AIA FILE No. 13-C

THE LALLY COLUMN

LALLY COLUMN COMPANY

SOLE MANUFACTURERS OF

GENUINE LALLY COLUMNS

BOSTON NEW YORK CHICAGO

AIA FILE No. 13-C

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ARCHITECTS SAMPLES
NEW YORK

LALLY COLUMNS

Structural Design and Details

MANUFACTURED BY THE LALLY COLUMN CO.
GENUINE
LALLY
COLUMN
TRADE MARK
BOSTON · NEW YORK · CHICAGO

A CATALOG for the assistance of architects, engineers and builders in designing Genuine Lally Columns in structures of steel, wood or concrete.

Special adaptations of the LALLY COLUMN and connections are covered by U. S. Patents:

2 033 595 2 439 129 2 469 455

Other patents pending.

LALLY COLUMN COMPANY
ESTABLISHED 1897
Sole Manufacturers of GENUINE LALLY COLUMNS

BOSTON
Erie and Albany Sts.
Cambridge 39, Mass.

NEW YORK
211 Lombardy St.
Brooklyn 22, N. Y.

CHICAGO
53 West Jackson Boulevard
Chicago 4, Ill.

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AT
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L15
1953

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LALLY COLUMN COMPANY
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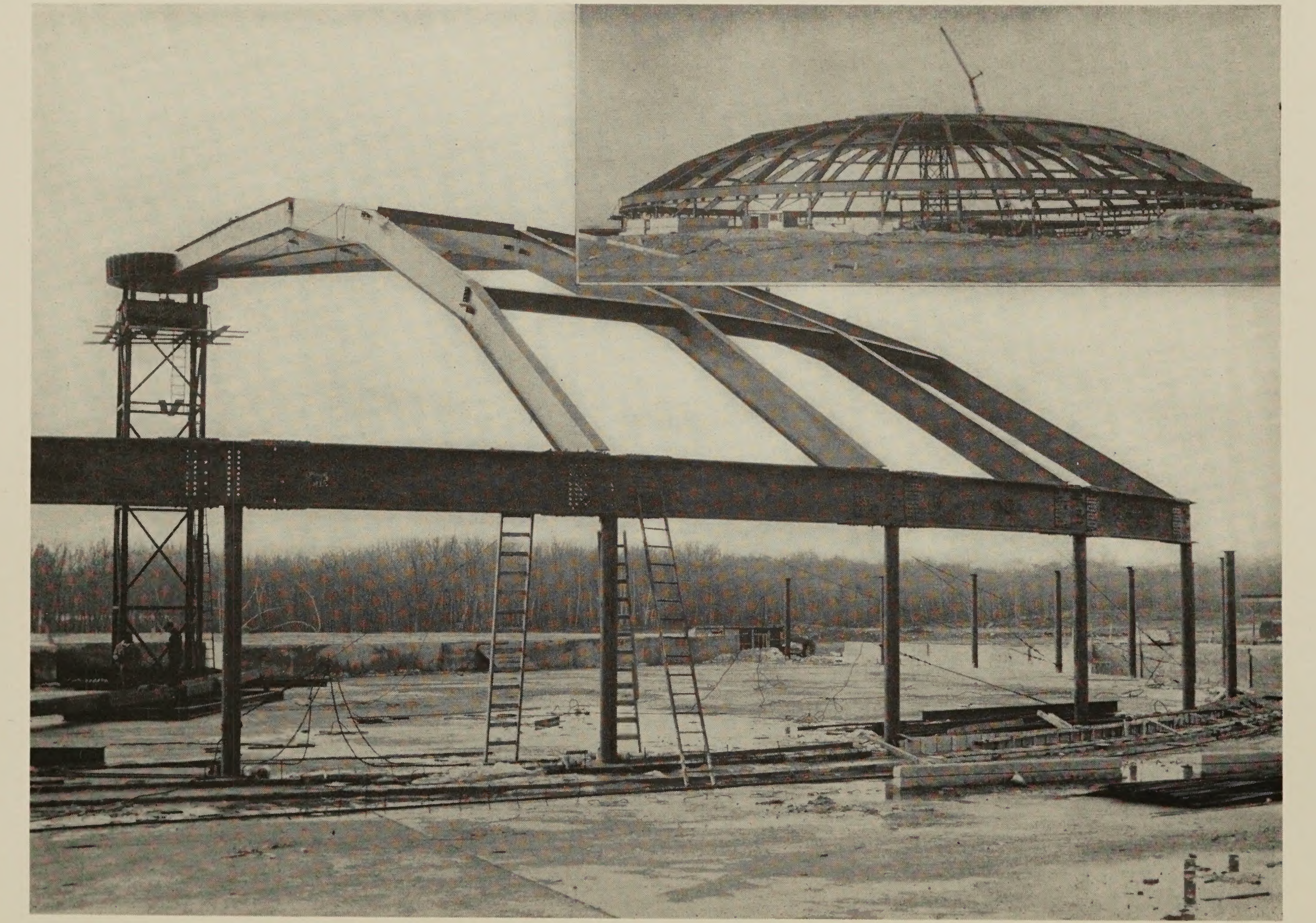
FOREWORD

THE LALLY COLUMN COMPANIES of Cambridge, Mass., Brooklyn, N. Y. and Chicago, Ill., are the sole manufacturers of GENUINE LALLY COLUMNS. All caps, bases and side connections were invented by Mr. John Lally, the founder of the company. The first tests on a prefabricated, concrete-filled pipe section were made by JOHN LALLY on Oct. 31, 1896 and the first U. S. Patent was issued on Nov. 22, 1898. The formulae used in the design of LALLY COLUMNS and connections have been developed by careful stress strain measurement and analysis of the test data, after extensive tests at Watertown Arsenal, Massachusetts Institute of Technology, Wentworth Institute and Columbia University. All official tests were witnessed and supervised by Architects, Engineers and Building Department Officials and approvals have been granted by all leading Building Departments in the U.S.A., for use of GENUINE LALLY COLUMNS. Their strength, fire resistance and practical performance have been demonstrated over a half-century of successful use in buildings of all classes of occupancy and in structural frames of steel, wood and concrete types.

The latest adaptation of the LALLY COLUMN is described in this catalogue as the LALLY EQUIDEPTH SYSTEM together with types of column heads for use in reinforced concrete floor construction.

The LALLY COLUMN COMPANY does not prepare engineering plans. Its endeavor will be to supplement the information in this catalogue by conference and advice of its ENGINEERING DIVISION in the solution of any problem involving the use of the LALLY COLUMN.



Middlesex Shopping Center, Framingham, Mass. Largest Arch-Rib Dome in the United States supported on genuine Lally columns.

CONTENTS

Page

Advisory Services 48
Architectural and Engineering Features 7

Base Plates 25
Brackets 25

Caps 25
Column Connections . . . . 14 to 20
Column Formula 10
Column Loads 10
Contents 5, 6

Details, Equidepth Head 32
    Thru-Web Plates 16, 27, 28
    Typical Shop 26
Dimensions and Properties of Pipe 9

Eccentric Load Factors 11
Eccentricity, Method of Calculation 12
Economy of Space 8
Equidepth Construction 29
    Column Design Analysis 40
    Column Head Design 40
    Formwork 47
    Frame Design and Analysis 39
    Head Types 30, 32, 33, 37, 38
    Inner Band Design 45
    Plate Heads 37, 38
    Plate Hinge Design 41
    Slab Thickness and Channel Head Sections 44

Foreword 3
Fireproofed Lally Columns 22, 23
Fireresistance 22

Illustrations
    Fig. 1—Space Economy 8
        2—Lightweight Columns 13
        3—Type A Construction 14
        4—Type B Construction 15
        5—Type C Construction 15
        6—Type D Construction 16
        7—Type D-1 Construction 17
        8—Type E Construction 18
        9—Type F Construction 19
        10—Type G Construction 20
        11—Pipe Reinforcement 21
        12—Angle and Rod Reinforcement 21
        13—Double Shell Columns 23
        14—Effects of Fire 24
        15—Typical Shop Details 26
        16—Exterior and Interior Heads 30
        17—Typical Equidepth Sections 31
        18—Skeleton Wall Construction 33
        19—Typical Channel Heads 33
        20—Hooper Holmes Insurance Bldg. 35
        21—Leopold Residence 36

Page
  - 22—Lally Plate Hinges 37
  - 23—Lally Plate Hinges 38
  - 24—Load-Distribution Diagram 39
  - 25—Column Band Calculation 42
  - 26—Column and Column Head Design 43
  - 27—Leopold Cantilever Slab 47
  - 28—Hooper Holmes Formwork 47
- Lally Order 29
- Lightweight Columns 13

- Materials of Manufacture 7, 13
- Mathematical Factors of Composite Section 11

- Patents 1
- Pipe Dimensions and Properties 9

- Reinforced Lally Columns 21

- Safe Loads
  - Base Plates 25
  - Brackets 25
  - Columns Lightweight 13
  - Columns Standard Heavyweight 10
  - Columns Extra Heavyweight 10
  - Thru-Web Plates 28
- Shop Details 26
- Specifications, Dimensions and Weights 7, 9, 13
  - Extra Heavyweight Columns 9
  - Lightweight Columns 13
  - Standard Heavyweight Columns 9

- Tables
  - Table 1—Heavyweight Columns, Dimensions and Weights 9
    - 2—Extra Heavyweight Columns, Dimensions and Weights 9
    - 3—Safe Loads Standard Heavyweight 10
    - 4—Safe Loads Extra Heavyweight 10
    - 5—Eccentric Load Factors Standard Heavyweight 11
    - 6—Eccentric Load Factors Extra Heavyweight 11
    - 7—Safe Loads Standard Brackets 25
    - 8—Safe Loads Standard Bases 25
    - 9—Web Plate Details 27
    - 10—Safe Loads Web Plate Connections 28
    - 11—Typical Equidepth Head Details 32
    - 12—Equidepth Head Sections and Slab Thickness 44
- Thru-Web Plates 16, 27, 28
- Types of Column Connections
  - Type A—Plain Cap and Base 14
    - B—Bracketed Cap (Single Story) 15
    - C—Bracketed Cap (Multi-Story) 15
    - D—Top Flange Tie and Thru-Web Plate 16
    - D-1—Double Splice and Thru-Web Plate 17
    - E—Double Flange Tie and Side Bracket 18
    - F—Standard Bracket and Thru-Web Tie 19
    - G—Bar Joist Construction 20

- Weight of Lally Columns 9

LALLY STANDARD HEAVYWEIGHT COLUMNS

ARCHITECTURAL AND ENGINEERING FEATURES

SAFETY:	The Lally Column Company, originators of Steel Shell Concrete-Filled Columns, are constantly checking the safety of their product thru laboratory tests. GENUINE LALLY COLUMNS carry a greater load than other type columns occupying the same floor area. See Fig. 1.
DURABILITY:	Fire, water and weight sufficient to destroy other type columns do not affect a LALLY COLUMN. They are the most durable building support made. See Fig. 14.
APPEARANCE:	GENUINE LALLY COLUMNS are especially adaptable to present-day architectural design. They present a neat, smooth, finished appearance and fit into partitions without unsightly projections.
MATERIALS:	GENUINE HEAVYWEIGHT LALLY COLUMN shafts are cut from mill tested steel pipe, A.S.T.M. Specification A-53. Caps, bases and connections are fabricated from Structural Steel Plate, A.S.T.M. Specification A-7, and are designed to suit any structural condition. LIGHTWEIGHT TYPE COLUMNS are manufactured from mill tested new steel tubing S.A.E. Specifications 1010. All shafts, caps, bases and connections are thoroughly tested and approved.
ECONOMY:	A substantial saving in field labor and materials is effected by using GENUINE LALLY COLUMNS. Expensive boxing required to finish other type columns is eliminated. In addition there is a considerable saving in floor space for the same load-bearing capacity.
EXTRA HEAVYWEIGHT:	LALLY COLUMNS can be manufactured from extra strong steel pipe. All columns are designated in the tables by actual outside diameter and weight per lineal foot of column. Larger columns in 14″ and 16″ sizes with a minimum shell thickness of 3/8″ are also available.
LIGHTWEIGHT:	LIGHTWEIGHT columns are also manufactured for use in basements of residential buildings as shown in Fig. 2, with concrete fill of the same mix as in LALLY HEAVYWEIGHT COLUMNS. They are furnished with steel plate or cast iron caps and bases.
WELDING:	In addition to the mechanical anchorage, all connections are welded in accordance with the standards of the American Welding Society and all welding is executed by licensed welders. Unless otherwise specified, all welds are 1/4″ standard bead or fillet.

LALLY COLUMNS SAVE FLOOR SPACE

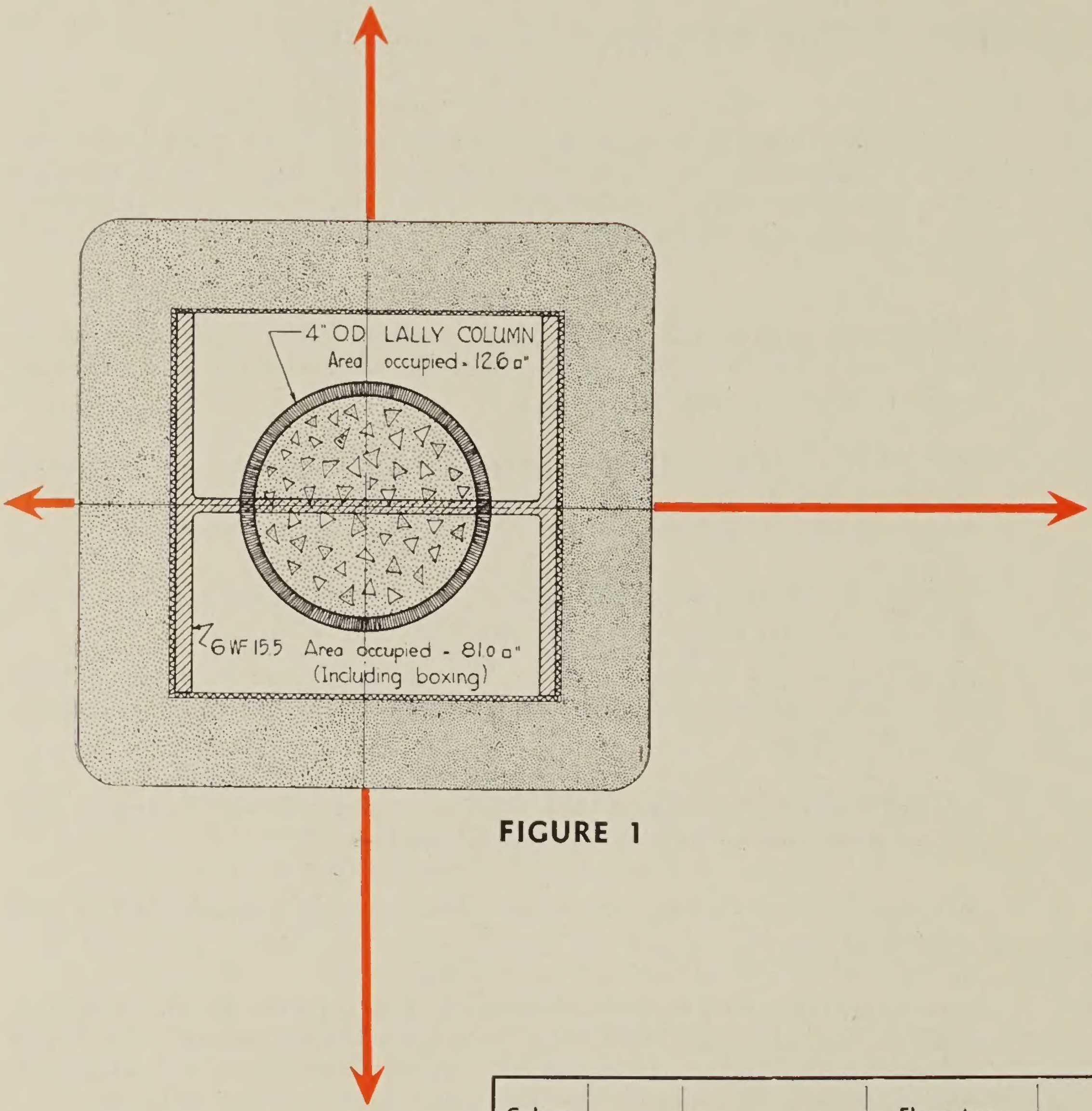


FIGURE 1

Calumn Load in Kips	Calumn Length	Required Calumn Size	Floor Area Occupied in Square Inches	Remarks
28	13′-0″	4″ O.D. LALLY 6″ WF 15.5	12.57 81.0 (Including baxing)	H Column boxed occupies over 6 times the areo.
70	20′-0″	6⅝″ O.D. LALLY 8″ WF 31	34.47 12.10 (Including boxing)	H Calumn baxed occupies over 3½ times the area.
140	20′-0″	8⅝″ O.D. LALLY 10″ WF 49	58.43 169.0 (Including baxing)	H Calumn boxed occupies over 2¾ times the area.

LALLY HEAVYWEIGHT COLUMNS

SPECIFICATIONS

Column Shaft—A.S.T.M.-A-53 mill tested black steel pipe, cut and slotted to receive beam connections.

Concrete Fill—Machine mixed and electrically agitated and compacted.

Proportions:
1 Part Portland Cement.
1½ Parts clean sharp sand.
3 Parts ½″ crushed stone or gravel.

Caps, Bases, Brackets and Web Plates—A.S.T.M.-A7 Steel Plate mechanically anchored and welded.

TABLE 1—Standard Heavyweight Columns

DIMENSIONS			ESSENTIAL DATA					
Outside Diameter of Calumn	Radius of Gyration (In Inches)		Inside Diameter af Pipe	Thickness af Metal	Area in Square Inches		Weight in Pounds Per Faot af Column	
Inches	Steel Only	Entire Section	Inches	Inches	Steel	Concrete	Steel Only	Entire Calum
3½″	1.2	1.7	3.068	0.216	2.23	7.39	7.575	15
4″	1.3	1.8	3.548	0.226	2.68	9.89	9.109	20
4½″	1.5	2.0	4.026	0.237	3.17	12.73	10.790	24
5″	1.7	2.2	4.506	0.247	3.69	15.95	12.538	29
5½″	1.9	2.4	5.047	0.258	4.30	20.01	14.617	36
6⅝″	2.2	2.8	6.065	0.280	5.58	28.89	18.974	49
7⅝″	2.6	3.1	7.023	0.301	6.92	38.74	23.544	64
8⅝″	2.9	3.5	7.981	0.322	8.40	50.03	28.554	81
9⅝″	3.3	3.8	8.941	0.342	9.97	62.79	33.907	100
10¾″	3.7	4.3	10.020	0.365	11.91	78.86	40.483	123
12¾″	4.4	4.8	12.000	0.375	14.58	113.10	49.562	169

TABLE 2—Extra Heavyweight Columns

DIMENSIONS			ESSENTIAL DATA					
Outside Diameter of Column	Radius of Gyration (In Inches)		Inside Diameter of Pipe	Thickness of Metal	Area in Square Inches		Weight in Pounds Per Faot af Column	
Inches	Steel Only	Entire Section	Inches	Inches	Steel	Concrete	Steel Only	Entire Column
6⅝″	2.2	3.2	5.761	0.432	8.405	26.07	28.57	56
8⅝″	2.9	4.0	7.625	0.500	12.760	45.66	43.39	91
10¾″	3.6	4.6	9.750	0.500	16.100	74.66	54.74	133
12¾″	4.3	5.2	11.750	0.500	19.240	108.44	65.42	178

ALWAYS SPECIFY the weight in pounds per foot of the column on plans as given in the last column of Tables I and 2. This is an added insurance that new steel pipe of the proper thickness will be furnished.

SAFE LOADS FOR LALLY
HEAVYWEIGHT COLUMNS (IN KIPS)

TABLE 3

Diam. of Column Inches	Weight Per Foot Lbs.	Area of Steel Sq. In.	Area of Concrete Sq. In.	UNBRACED LENGTH OF COLUMN—IN FEET															Max. Length in Feet
				6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	
3½	15	2.23	7.39	38	35	32	29	27	24										11.64
4	20	2.68	9.89	49	46	43	40	37	34	31	28								13.37
4½	24	3.17	12.73	62	59	55	52	49	46	42	39	36	33						15.10
5	29	3.69	15.96	76	72	69	65	62	58	55	51	48	44	41	37				16.83
5½	36	4.30	20.01	92	88	85	81	77	73	70	66	62	58	55	51	47	43		18.78
6⅝	49	5.58	28.89	128	124	120	116	112	108	103	99	95	91	87	83	78	74	70	22.45
7⅝	64	6.92	38.74	166	161	157	152	148	143	139	134	130	125	121	116	111	107	102	25.92
8⅝	81	8.40	50.03	211	206	201	196	191	186	181	176	171	166	161	156	151	146	141	29.38
9⅝	100	9.97	62.79	259	254	249	243	237	232	227	221	216	210	205	199	194	189	183	32.84
10¾	123	11.91	78.86	319	313	307	301	295	289	284	278	272	266	260	254	248	242	236	36.74
12¾	169	14.58	113.10	422	415	409	402	396	389	383	376	370	363	357	350	344	337	331	43.77

TABLE 4 • EXTRA HEAVYWEIGHT COLUMNS

Diam. of Column Inches	Weight Per Foot Lbs.	Area of Steel Sq. In.	Area of Concrete Sq. In.	UNBRACED LENGTH OF COLUMN—IN FEET																Max. Length in Feet
				10	11	12	13	14	15	16	17	18	19	20	24	28	32	36	40	
6⅝	56	8.41	26.07	148	143	137	132	126	121	115	110	104	99	93						21.95
8⅝	91	12.76	45.66	250	244	239	232	225	219	212	206	199	192	185	157	129				28.78
10¾	133	16.10	74.66	356	349	342	335	328	321	314	306	298	292	285	255	225	197	169		36.28
12¾	178	19.24	108.44	468	460	452	444	436	428	420	412	405	397	389	360	328	296	264	232	43.35

LOAD FORMULA

The following formula is used in computing the safe carrying capacity of LALLY COLUMNS:—

$$P = (Ac + 12\,As)\,(1{,}600 - 24\,l/d)$$

where

P = Safe carrying capacity in pounds.
Ac = Area of concrete in square inches.
As = Area of steel in square inches.
l = Length of column in inches.
d = Diameter of column in inches.

Limit of length = 40 diameters ($l/r = 120$ for steel pipe only).

For radius of gyration of the equivalent composite section see Tables 1 and 2.

The safe loads in Tables 3 and 4 are given in thousands of pounds and are based on all loads being applied axially.

For determining the strength of LALLY COLUMNS under eccentric loads, see Tables 5 and 6 and text on following pages.

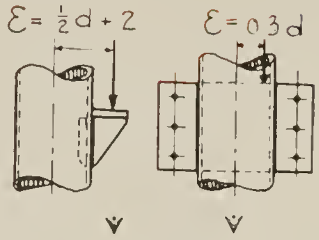
NOTE

Always specify on plans weight per foot of column length as shown in Tables 3 and 4.

ECCENTRIC LOADS ON LALLY COLUMNS

TABLE 5 • STANDARD HEAVYWEIGHT COLUMNS (H.W.)

Data for computing the strength of LALLY COLUMNS under eccentric loads.



1	2	3		4	5	6	7	8
DIAMETER of Column in Inches	% Steel	RADIUS OF GYRATION		MOMENT OF INERTIA	SECTION MODULUS	EQUIVALENT DIRECT LOAD to be added		
		Steel Section	Combined Section	I	S	10,000 in lbs. Bending Moment	Standard Bracket	Thru Plate
3½	30	1.2	1.7	3.75	2.13	13.3	5.0	1.5
4	27	1.3	1.8	5.90	2.95	11.9	4.8	1.5
4½	25	1.5	2.0	8.87	3.94	10.8	4.6	1.5
5	23	1.7	2.2	12.98	5.19	9.8	4.3	1.5
5½	22	1.9	2.4	18.88	6.78	8.3	4.1	1.5
6⅝	19	2.2	2.8	35.25	10.65	7.4	3.9	1.5
7⅝	18	2.6	3.1	58.48	15.33	6.7	3.8	1.6
8⅝	17	2.9	3.5	91.90	21.16	5.8	3.7	1.6
9⅝	16	3.3	3.8	132.67	27.57	5.4	3.7	1.6
10¾	15	3.7	4.3	207.75	38.64	4.9	3.6	1.6
12¾	13	4.4	4.8	361.50	56.70	4.2	3.5	1.6

TABLE 6 • EXTRA HEAVYWEIGHT COLUMNS (E.H.W.)

1	2	3		4	5	6	7	8
DIAMETER of Column in Inches	% Steel	RADIUS OF GYRATION		MOMENT OF INERTIA	SECTION MODULUS	EQUIVALENT DIRECT LOAD to be added		
		Steel Section	Combined Section	I	S	10,000 in lbs. Bending Moment	Standard Bracket	Thru Plate
6⅝	33	2.1	3.3	45.24	13.7	7.7	4.1	1.5
8⅝	28	2.7	4.0	119.88	27.8	6.0	3.8	1.6
10¾	22	3.3	4.6	249.18	46.3	4.8	3.5	1.6
12¾	18	4.0	5.2	438.92	68.9	4.1	3.4	1.6

NOTES:

1. The Moment of Inertia and Section Modulus given in columns 4 and 5 of Tables 5 and 6 are based on the steel equivalent of the combined section. To express the Moment of Inertia in terms of the concrete section multiply by 12.

2. For vertical loads supported directly on top of the column, the eccentricity may be considered zero and the loads concentrically applied.

3. For each 10,000 inch pounds of unbalanced moment on the column add the number of kips shown in column 6 to the sum of all the vertical loads, or

4. Add the number of kips in columns 7 or 8, depending upon the type of connection used, for each kip of unbalanced load, to the sum of all the vertical loads, or

5. For average results use the factor 4 times the unbalanced vertical load to be added to the sum of all the vertical loads when standard brackets are used, or use the factor of 1½ when thru-plate connections are used.

6. For beams framing at right angles to the thru-plate, the eccentricity is measured from the center of column to point of application of the load as shown in case "D." The equivalent direct load for each 10,000 inch pounds bending moment can be computed from the factors in Column 6 of Tables 5 and 6.

ECCENTRIC LOADS ON LALLY COLUMNS

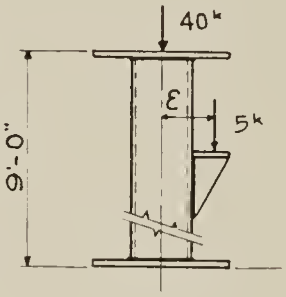
SAMPLE COMPUTATIONS for DESIGN of LALLY COLUMNS with ECCENTRIC LOADS

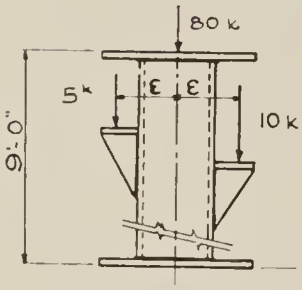
The moment in a framed bent resulting in an eccentric load on a column is dependent upon the relative stiffness of the beam and the column forming the frame and the stiffness of the connections. In the table on the preceding page the equivalent load due to one kip, eccentrically applied on the column, is given for the several types of connections. With the standard bracket connection, the eccentricity varies more with the size of the column than in the case of a thru-plate connection.

The following problems have been solved using the average factors given in note 5 on the preceding page. Having determined the preliminary design by the average factor method, the engineer can check his design by a more accurate analysis of the framed bent using the predetermined sizes of beams and columns.

Case A Standard Bracket, Single Eccentric Load

Direct Load $= 40^k+5^k=45^k$
Eccentric Load $= 5^k$
Equivalent Direct Load $= 5^k \times 4 = 20^k$
" Total Load $= 45^k+20^k=65^k$
Requires $5''\phi$ H.W. LALLY COLUMN.
Actual Factor for $5''\phi$ Col. is 4.3, Equiv. to $5^k \times 4.3 = 21.5^k$



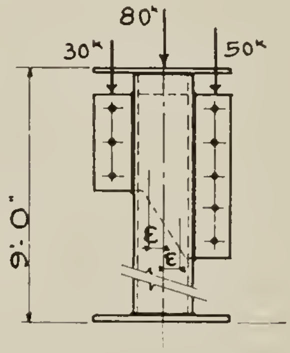


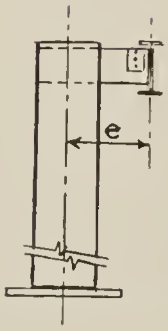
Case B Standard Bracket, 2 Eccentric Loads

Direct Load $= 80^k+10^k+5^k=95^k$
Resultant Eccentric Load $= 10^k-5^k=5^k$
Equivalent Direct Load $= 5^k \times 4 = 20^k$
" Total Load $= 95^k+20^k=115^k$
Requires $6\frac{5}{8}''\phi$ H.W. LALLY COLUMN.
Actual Factor for $6\frac{5}{8}''\phi$ Col. is 3.9, Equiv. to $5^k \times 3.9 = 19.5^k$

Case C Thru-Plate, 2 Eccentric Loads

Direct Load $= 80^k+50^k+30^k=160^k$
Resultant Eccentric Load $= 50^k-30^k=20^k$
Equivalent Direct Load $= 20^k \times 1.5 = 30^k$
" Total Load $= 160^k+30^k=190^k$
Requires $8\frac{5}{8}''\phi$ H.W. LALLY COLUMN.
Actual Factor for $8\frac{5}{8}''\phi$ Col. is 1.6, Equiv. to $20^k \times 1.6 = 32^k$





Case D Thru-Plate With Load at Right Angles

Determine moment in inch-pounds with eccentricity e and transform into equivalent direct load by applying factors given in Column 6 of Tables 5 and 6.

LALLY LIGHTWEIGHT TYPE COLUMNS

IN 3½″ and 4″ (For Residential Construction)

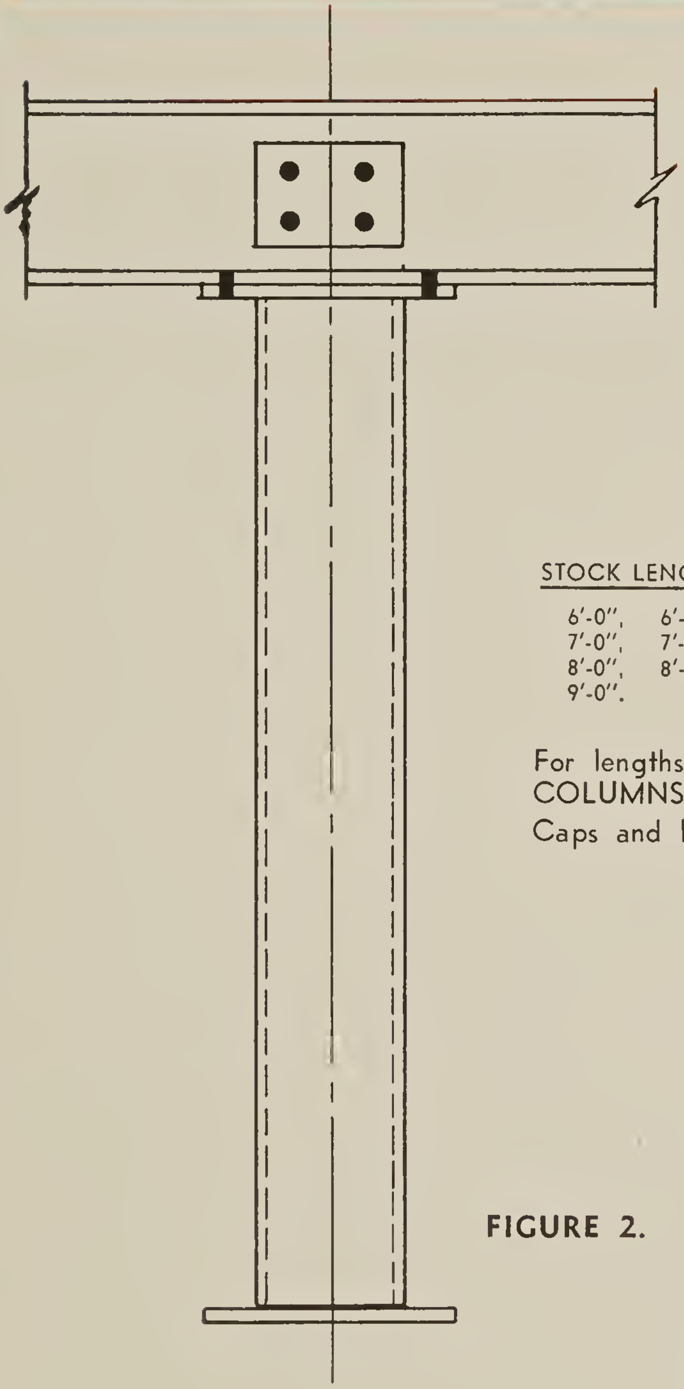


FIGURE 2.

STOCK LENGTHS	STOCK CAPS	STOCK BASES
6′-0″, 6′-6″,	4″ x ¼″ x 8″	6″ x ¼″ x 6″
7′-0″, 7′-6″,	6″ x ¼″ x 6″	8″ x ¼″ x 8″
8′-0″, 8′-6″,	6″ x ¼″ x 8″	
9′-0″.		

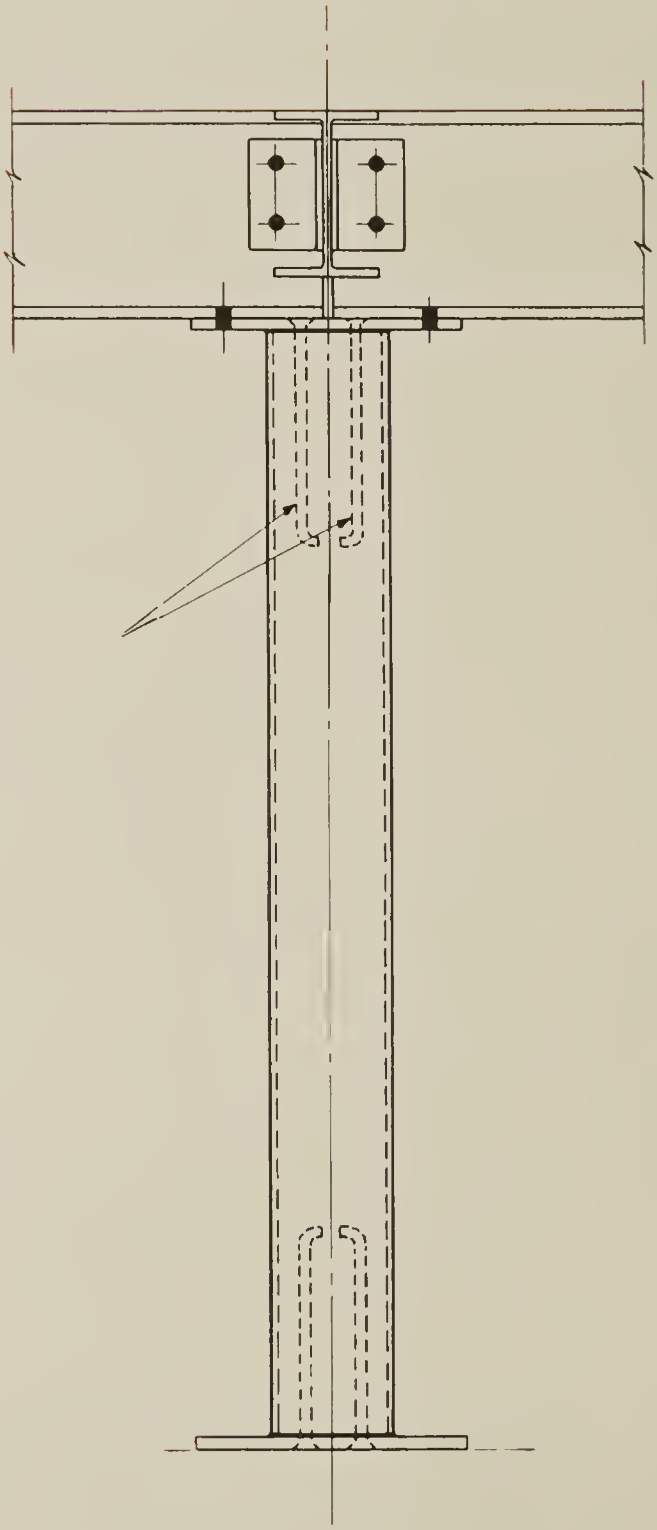
For lengths over 9′-0″, always use HEAVYWEIGHT LALLY COLUMNS.

Caps and bases furnished loose or welded to column shaft.

LIGHTWEIGHT TYPE COLUMNS are made in both 3½″ and 4″ O.D. and are manufactured as a cellar column to support small I-Beams or wood girders in RESIDENTIAL CONSTRUCTION ONLY. This type column is not made for cellar supports in Apartment, Store or Industrial buildings.

LIGHTWEIGHT TYPE COLUMNS are manufactured under the same strict supervision as Lally Heavyweight Columns. The column shaft is high carbon mill tested new steel tubing—Specifications SAE 1010. Caps and bases are steel plate A.S.T.M.—A-7 or Cast Iron. The concrete mix is the same as used in heavyweight columns.

LALLY HEAVYWEIGHT COLUMNS—Type-A



TYPE A

Standard Cap and Base

Countersunk head bolts embedded in concrete fill and welded to plates.

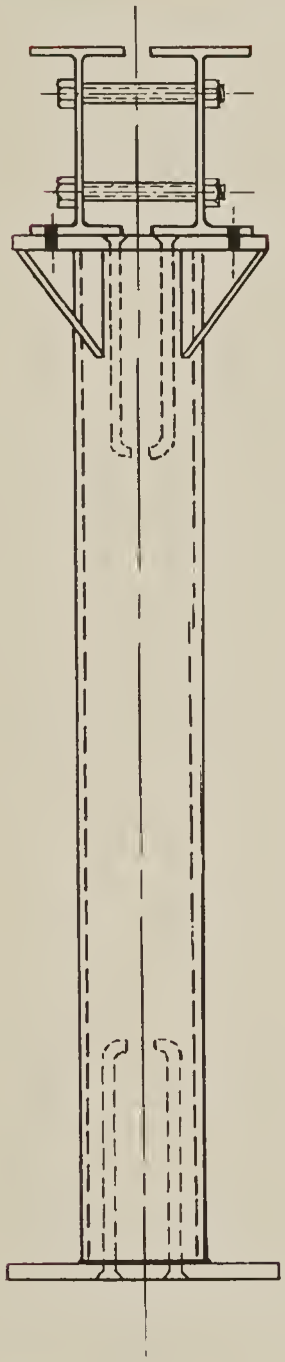
Structural steel caps and bases electrically welded in addition to mechanical anchorage.

Specify:

"All Steel Shell Concrete - Filled Columns Shown on Plans Shall Be Genuine LALLY Columns Made by the Lally Column Company."

LALLY HEAVYWEIGHT COLUMNS—Types B and C

FIGURE 4.



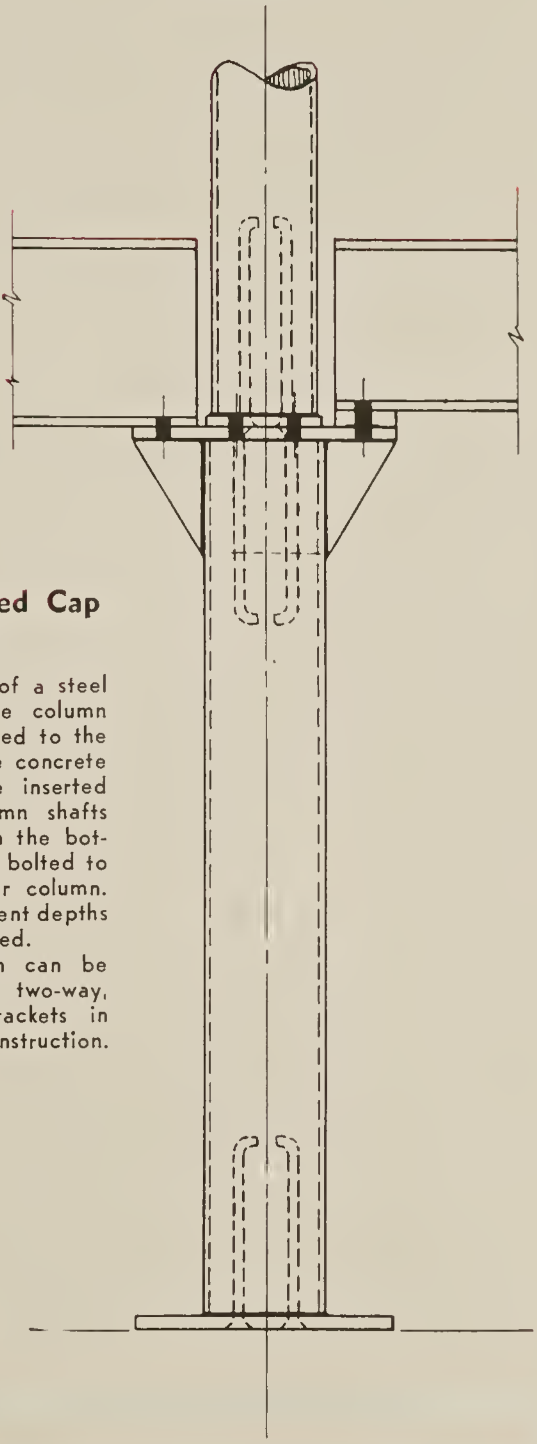
TYPE B

Bracketed Cap Construction

Structural steel brackets are mechanically anchored inside of the column shaft.

Caps, bases and brackets are electrically welded in addition to the mechanical anchorage.

FIGURE 5.



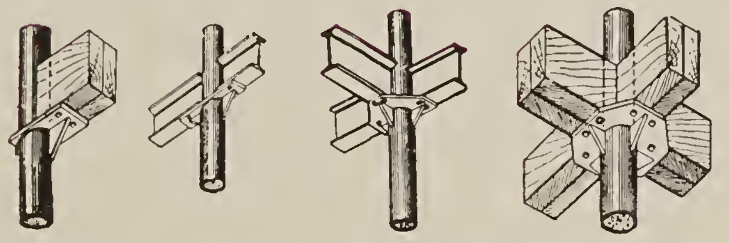
TYPE C

Multi-Story Bracketed Cap Construction

This connection consists of a steel cap plate fastened to the column with countersunk bolts welded to the plate and embedded in the concrete filling. Steel brackets are inserted through slots in the column shafts and a steel splice plate on the bottom of the column above is bolted to the cap plate of the lower column.

Where beams are of different depths necessary fillers are furnished.

This type of connection can be furnished with one-way, two-way, three-way or four-way brackets in one-story and multi-story construction.



One-Way Two-Way Three-Way Four-Way

Above are shown various caps for wood and steel beams.

TYPE D

Multi-Story Top Flange and Web Plate Construction

Where a particularly rigid connection between steel floor beams and LALLY COLUMNS is desired, the "thru-plate" design is recommended. The webs of the floor beams are secured to the thru-plate of the column with sufficient bolts or rivets to carry the load. The plate is made sufficiently thick to safely resist the bending moment, torsion and resulting gross shear. See Table 10.

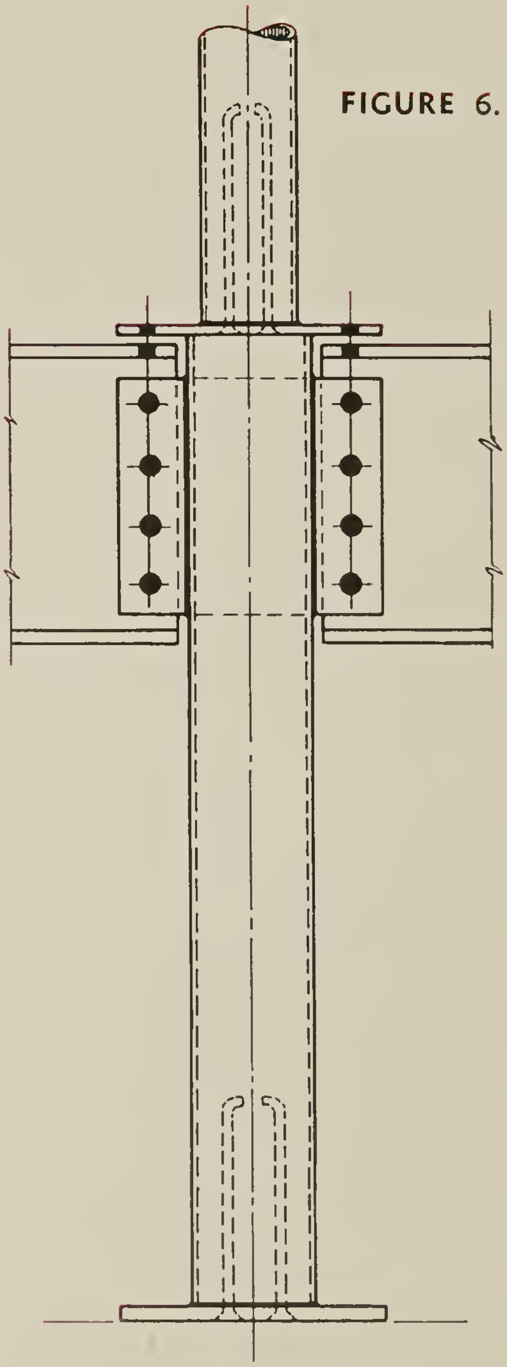
The "thru-plate" is welded to the column along all lines of contact.

The "thru-plate" design can be used advantageously where a finished ceiling is placed directly under the bottom flange of the floor beams to avoid the projection of the ribs of LALLY brackets below the ceiling line.

Columns of this type are manufactured in either one- or two-story lengths.

In this type of construction the base plate of the upper column is bolted to the top flange of the beams below, or a double splice plate can be furnished as shown in Type D-1. All necessary holes are punched in the plates before shipment to the site.

FIGURE 6.



MULTI-STORY
CONSTRUCTION

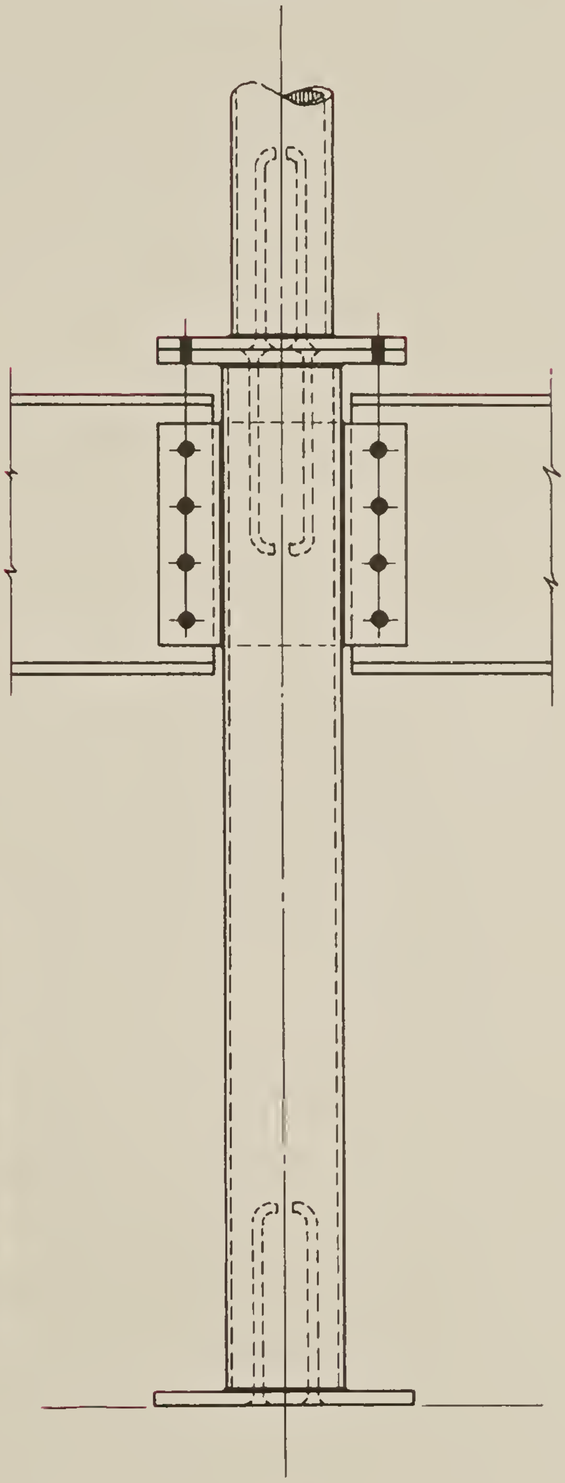
LALLY HEAVYWEIGHT COLUMNS—Type D-1

TYPE D-1

Multi-Story Double Splice Plate Construction

This detail is a variation of that shown on the preceding page in that the base plate of the upper column is bolted to a cap plate on the lower column rather than to the top flanges of the floor beams. The columns are fabricated in one- or two-story lengths as illustrated.

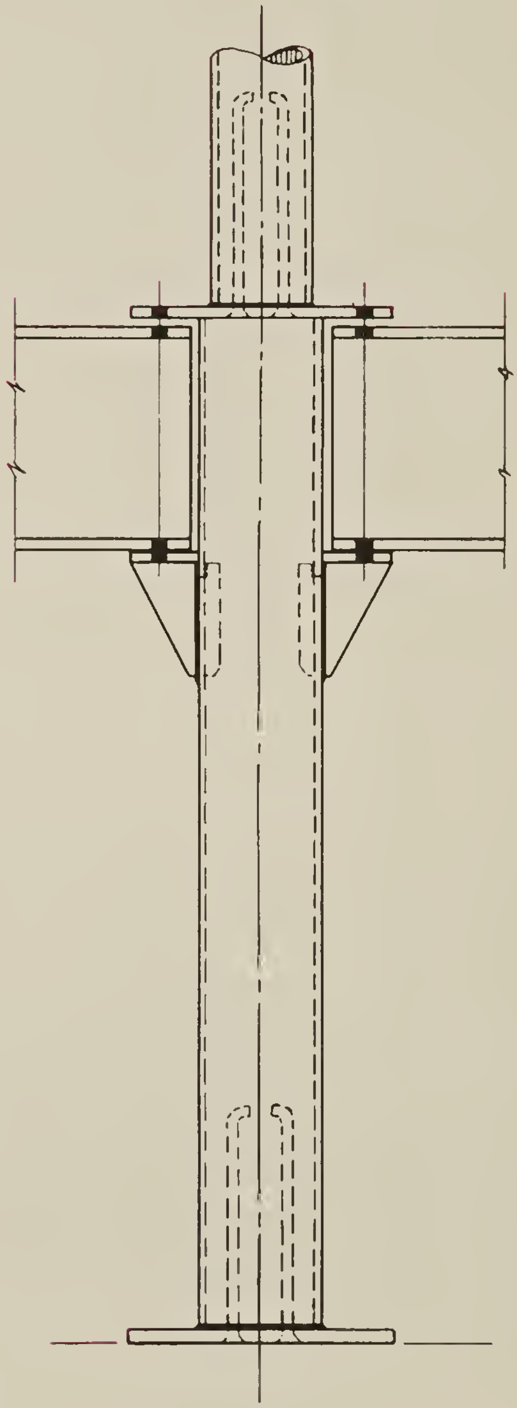
FIGURE 7.



Typical use of multi-story Lally Columns for two-story apartment and store construction.

LALLY HEAVYWEIGHT COLUMNS—TYPE-E

FIGURE 8



TYPE E

Multi-Story Double Flange Tie and Side Bracket Contruction

This detail offers a strong and rigid connection where double flange tie construction is required.

As in other types all bases and side brackets are continuously welded to the column shaft along all lines of contact in addition to the mechanical anchorage.

For special conditions or greater load capacity, special brackets can be designed and furnished.

TYPE F

Standard Bracket and Web Tie Construction

Standard brackets at different elevations with web ties and bolted splice plates.

Caps, bases, side brackets and web tie plates are electrically welded in addition to the mechanical anchorage.

Manufactured in one- or two-story lengths for multi-story construction.

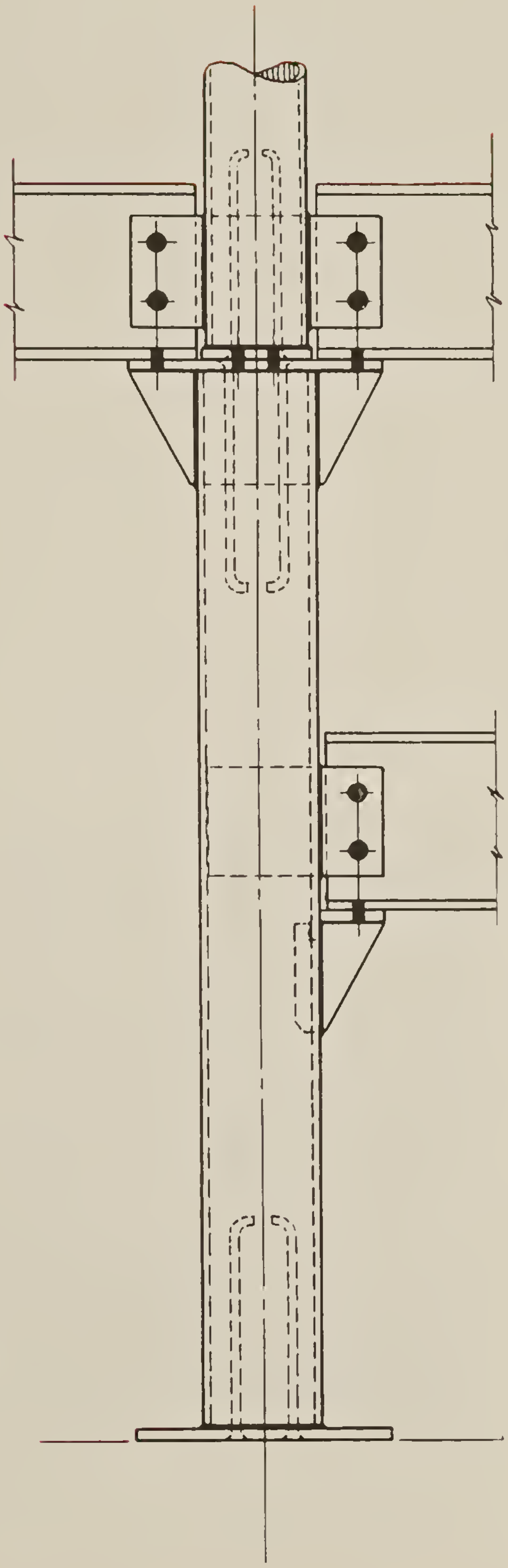
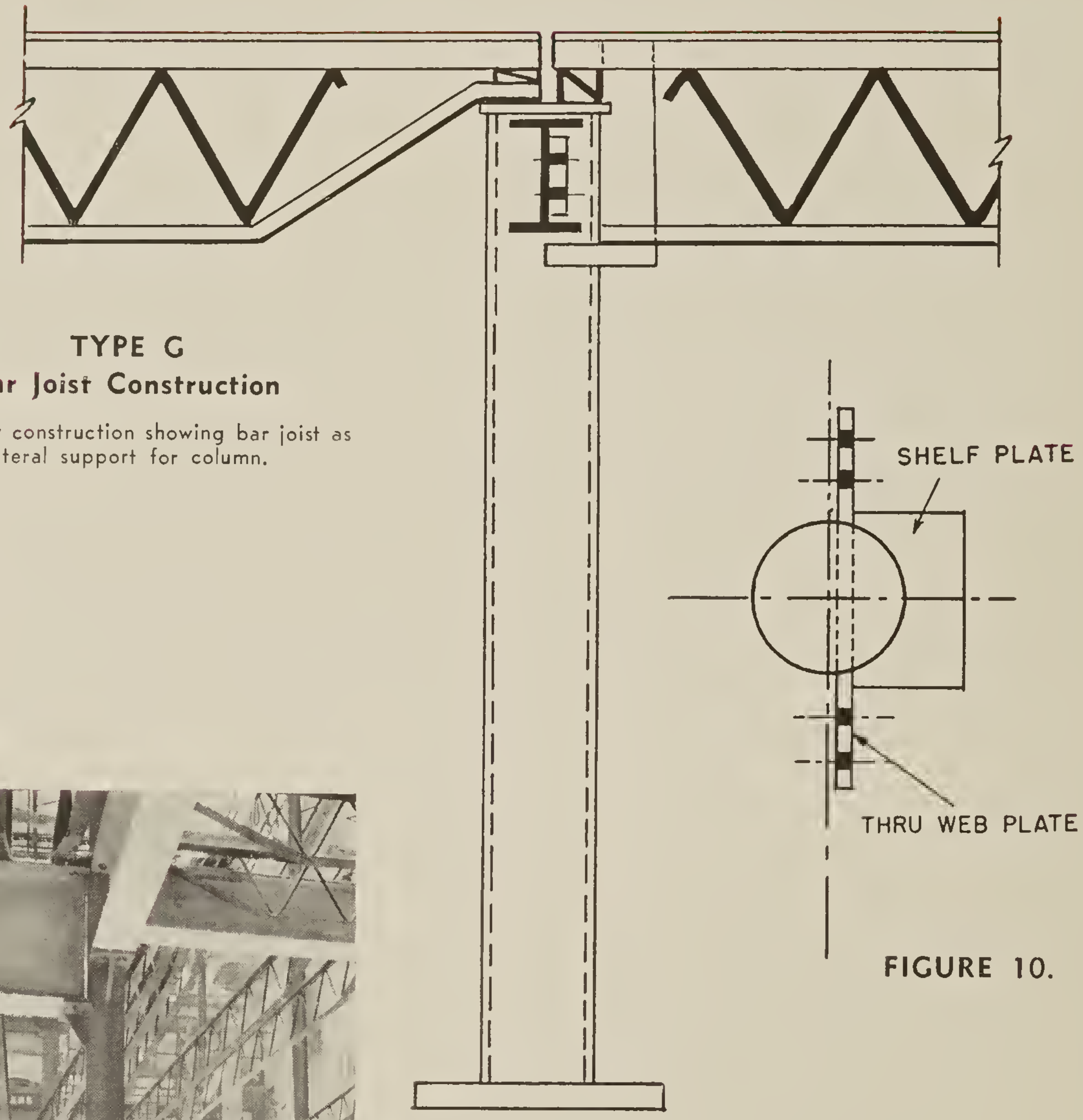


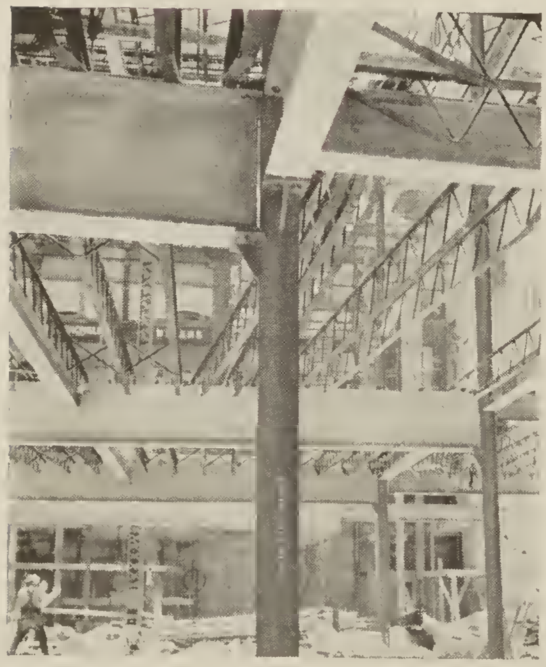
FIGURE 9.



TYPE G
Bar Joist Construction

One-story construction showing bar joist as lateral support for column.

FIGURE 10.



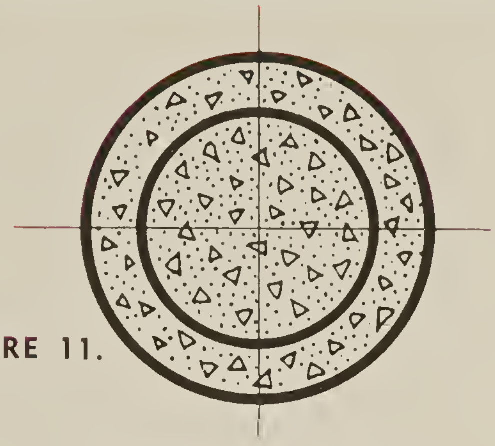
Genuine LALLY COLUMNS supporting steel girders and bar joists for multi-story construction.

REINFORCED LALLY COLUMNS

To increase the safe load capacity of a LALLY COLUMN, steel rod, pipe or angle reinforcement may be embedded in the concrete as shown in the accompanying sketches, or extra-heavy, strong pipe may be used. The use of extra-heavy pipe will substantially increase the safe carrying capacity of a LALLY COLUMN as shown in Table 4.

These methods of reinforcement are valuable where the diameter of the column must be limited, as, for example, to eliminate pilaster projections in partitions or walls.

FIGURE 11.



REINFORCED LALLY COLUMN
With Pipe Reinforcement
The difference in diameters of the pipes should be a minimum of four inches.

The reinforcement is cut and placed to achieve full bearing on cap and base plates.

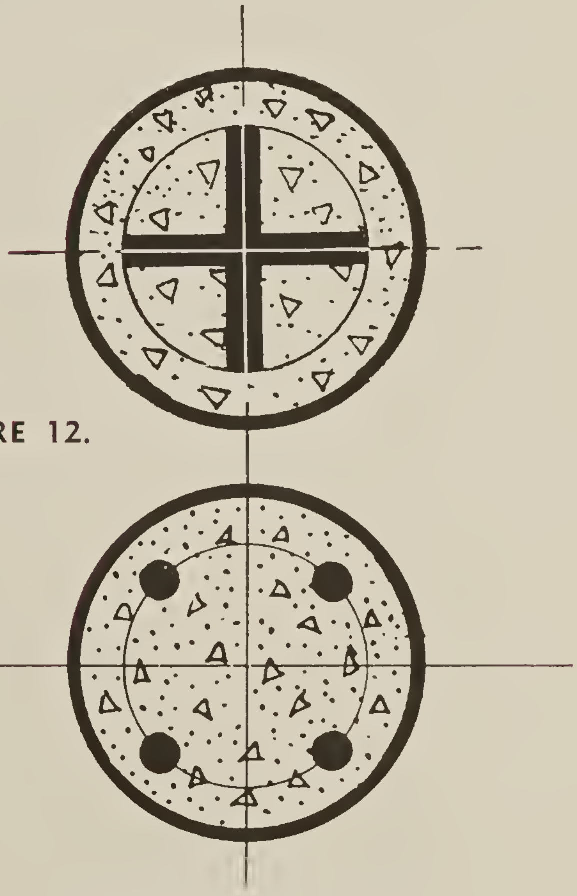
Safe loads for reinforced LALLY COLUMNS are computed by the same formula as for unreinforced columns shown on page 10. In this formula A_s represents the total area of outer shell plus the reinforcement.

Example: Find the safe load for a $10\frac{3}{4}''\phi$ column, 16 ft. long, reinforced with a $6\frac{5}{8}''\phi$ pipe.

$A_c = 78.86 - 5.58 = 73.28\square''$, $A_s = 11.91 + 5.58 = 17.79\square''$

$$P = (73.28 + 12 \times 17.79)\left(1{,}600 - 24 \times \frac{16 \times 12}{10.75}\right) = 331.6 \text{ kips.}$$

FIGURE 12.



REINFORCED LALLY COLUMNS
With Angle or Rod Reinforcement
The diameter of the circle on which the rods are located should be two inches less than the diameter of the pipe.

FIRERESISTANCE OF LALLY COLUMNS

ALTHOUGH controlled fire tests under standard time-temperature procedure have developed fireresistance ratings of ¾ to 1 hour for the unprotected LALLY COLUMN (Underwriters Laboratories, Chicago 1917-1919) actual fire experience demonstrates even superior fire protection properties. Without additional exterior protection, the LALLY COLUMN has withstood the damaging effects of intense conflagrations, as in the Chelsea Fire of April, 1908, the Salem Fire of June, 1914 (both in Massachusetts), the Edison Plant Fire at West Orange, N. J. in December, 1914, and the Standard Oil Fire in Greenpoint, N. Y., in September, 1919. See Fig. 14. In many instances, structural steel frames have distorted and failed, cast-iron columns have broken and even melted, exposed concrete has fused in the same structure in which the LALLY COLUMNS remained standing.

In controlled fire tests, the fireresistance ratings of LALLY COLUMNS have generally been determined by rise in temperature on the exterior of the steel shell to the theoretical end point of 1000° F. The number of official tests to date are inadequate to determine the real fire characteristics of this type of column. Expanded volcanic glass, furnace slag, pumice, mica and coverings of the various modern lightweight concrete and plaster compositions can be applied to the concrete-filled pipe column on metal or gypsum lath at the site to develop ratings of 1½, 2, 3 or 4-hour fireresistance if desired. Fireproofed columns can also be furnished with a double shell and a one inch or more annular space filled with perlite or vermiculite concrete to provide 3 and 4-hour resistance. See Fig. 13.

The ¾-hour fireresistance of the unprotected LALLY COLUMN is sufficient to withstand the inherent fire hazard of buildings of the residential use group as now recognized in the Basic Building Code promulgated by the Building Officials Conference of America, Inc., 1950 Edition, without any further protection. It is generally conceded that the probable intensity of fire in buildings for dwelling purposes does not exceed the equivalent severity of the standard controlled ¾-hour fire test procedure. The ¾-hour fireresistance rating has also been adopted in many of the more recent building codes, including Cleveland (Ohio), and the proposed New York, New Jersey and Connecticut State Codes.

LALLY DOUBLE-SHELL FIRE PROOF COLUMNS

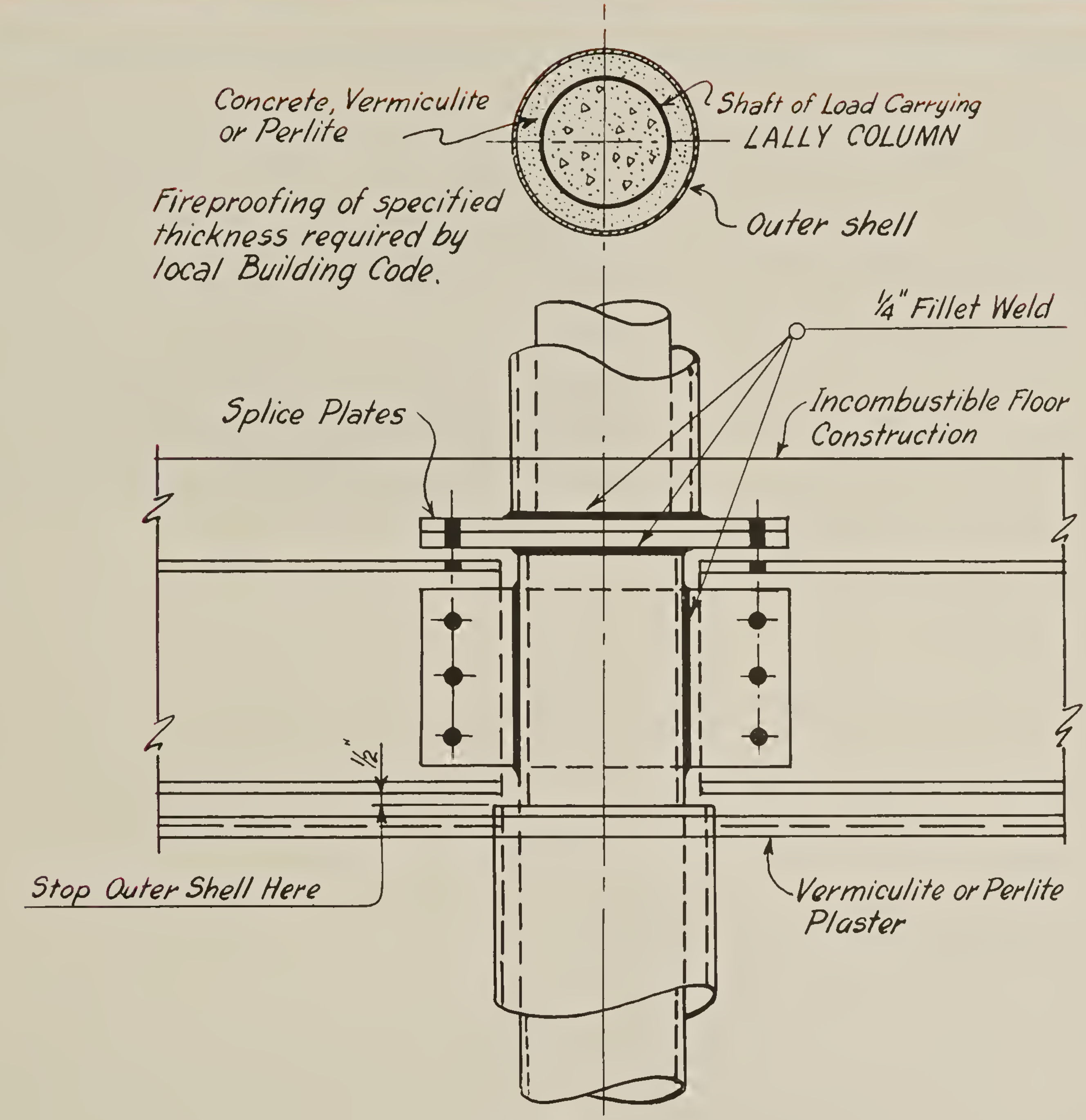

Concrete, Vermiculite or Perlite
Shaft of Load Carrying LALLY COLUMN
Fireproofing of specified thickness required by local Building Code.
Outer shell
¼" Fillet Weld
Splice Plates
Incombustible Floor Construction
½"
Stop Outer Shell Here
Vermiculite or Perlite Plaster

FIGURE 13.

Manufactured to meet any degree of fireresistance rating or thickness of concrete protection.

Outer shell and concrete or vermiculite fireproofing between outer shell and inside column shaft are designed FOR FIREPROOFING PURPOSES ONLY.

The inner column and the concrete inside this inner column are the load carrying members.

Without additional exterior protection, unprotected LALLY COLUMNS have a fire resistance rating three times greater than unprotected structural steel.

If desired, outer shells of less thickness than the standard pipe can be used for fireproofing purposes.

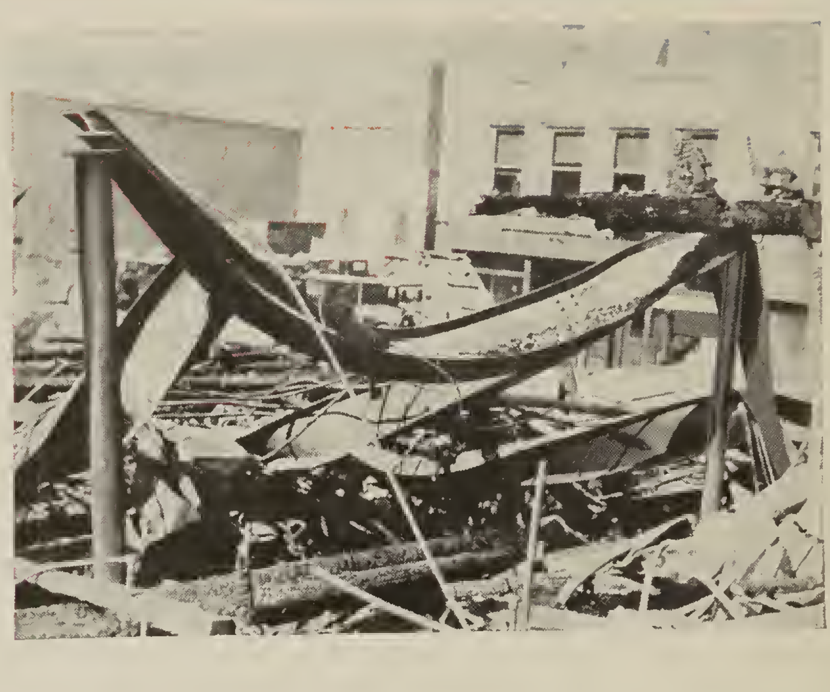
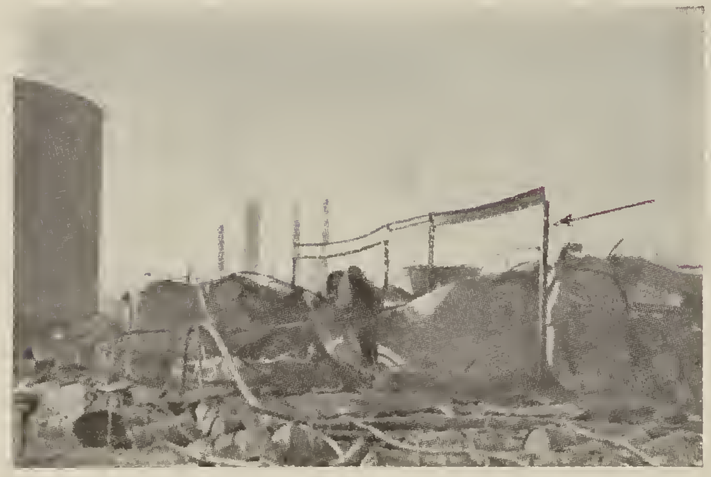


FIGURE 14

EXPERIENCE PROVES GENUINE LALLY COLUMNS PROVIDE GREATER FIRE RESISTANCE THAN STRUCTURAL STEEL

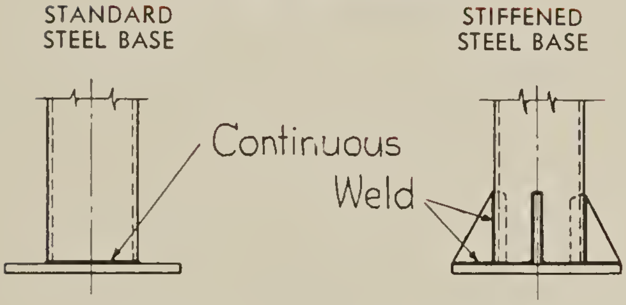


EFFECTS OF FIRE WHICH RAZED THE STANDARD OIL PLANT AT BROOKLYN, N. Y., LEAVING LALLY COLUMNS STILL STANDING.

STANDARD BASES AND CAPS FOR GENUINE LALLY COLUMNS

TABLE 8

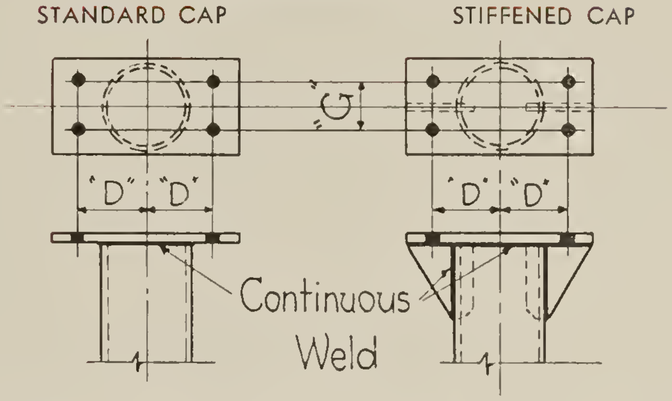
STANDARD BASES



Safe loads are based on 500 pounds per square inch bearing on concrete.

Column Diameter	Size of Base Plate	Safe Load in Kips	Thickness of Base Plate		Number of Brackets for Stiffened Base
			Standard Base	Stiffened Base	
3½	8 x 8	32.0	⅝	½	4
4	9 x 9	40.5	¾	½	4
4½	10 x 10	50.0	⅞	½	4
5	12 x 12	72.0	1	½	4
5½	14 x 14	98.0	1¼	¾	4
6⅝	16 x 16	128.0	1¼	¾	4
7⅝	18 x 18	162.0	1½	¾	8
8⅝	20 x 20	200.0	1¾	¾	8
9⅝	22 x 22	242.0	1¾	¾	8
10¾	24 x 24	288.0	2	⅞	8
12¾	28 x 28	392.0	2¼	⅞	8

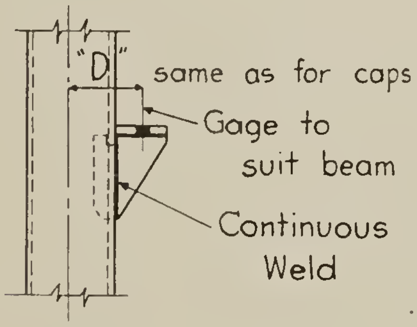
STANDARD CAPS



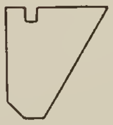
Column Diameter	Distance "D"		Thickness of Cap Plate
	Standard Cap	Stiffened Cap	
3½	3¼	4¼	½
4	3½	4½	½
4½	3¾	4¾	½
5	4	5	⅝
5½	4¼	5¼	⅝
6⅝	4¾	6¼	¾
7⅝	5	6¾	¾
8⅝	5¾	7¼	¾
9⅝	6¼	7¾	¾
10¾	7	8¼	¾
12¾	8	9	¾

NOTES: — 1. Distance "G" equals the standard gage of the supported beam. 2. The width of plate is determined by the beam flange. 3. Distance "D" is constant for 1, 2, 3 and 4-way caps. 4. Caps may be used for steel, wood or concrete construction.

STANDARD SIDE BRACKETS



All brackets for bases, caps and side brackets are notched as shown to provide mechanical anchorage in addition to ¼" continuous welding on all lines of contact.



For special brackets consult our Engineering Department.

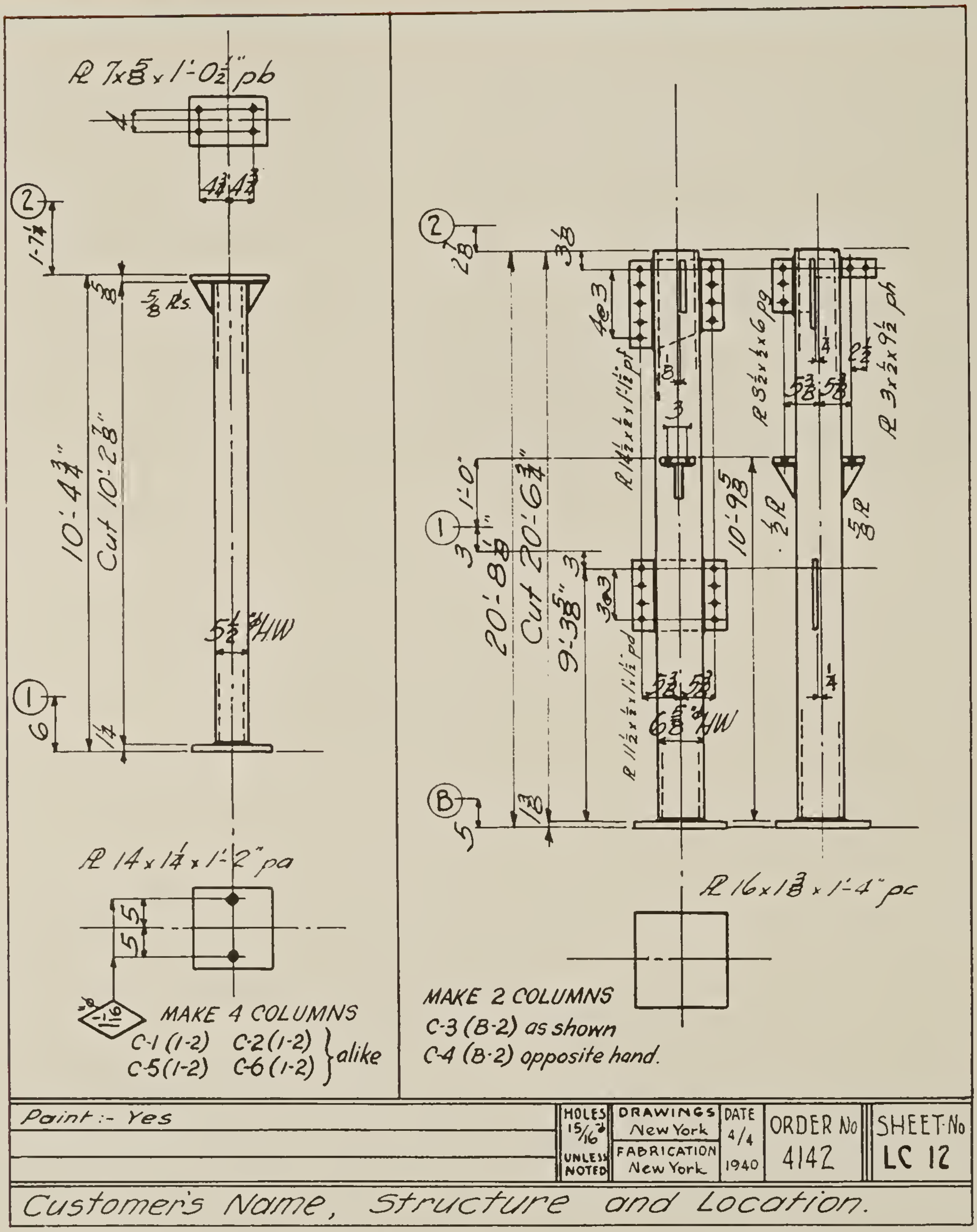
TABLE 7

SAFE WORKING LOADS for STANDARD BRACKETS—(SAFETY FACTOR OF 4)

LOAD in POUNDS	NUMBER of BRACKETS	THICKNESS of VERTICAL PLATE
To 16,000	1	½
To 21,000	1	⅝
To 30,000	1	¾
To 42,000	2	2 at ⅝
To 60,000	2	2 at ¾

TYPICAL SHOP DETAILS for LALLY COLUMNS

FIGURE 15



A typical shop drawing is illustrated, giving the essential data necessary for the fabrication of LALLY COLUMNS.

The important data are as follows:

1. Overall length.
2. Cut length of pipe.
3. Diameter.
4. The location of all holes or bearing surfaces relative to the end and center line of the pipe and for reference to the floor lines.

It is not necessary to show the bolts with countersunk heads which are used to provide mechanical anchorage for base and cap plates as these bolts will be provided whether called for or not.

In addition 1/4" continuous welds are always provided for caps, bases, brackets and thru-plates along all lines of contact with the pipe.

LALLY COLUMN WEB-PLATE CONNECTIONS

TYPICAL THRU-WEB PLATE DETAIL

TYPE 1
SINGLE LINE OF HOLES

TYPE 2
DOUBLE LINE OF HOLES

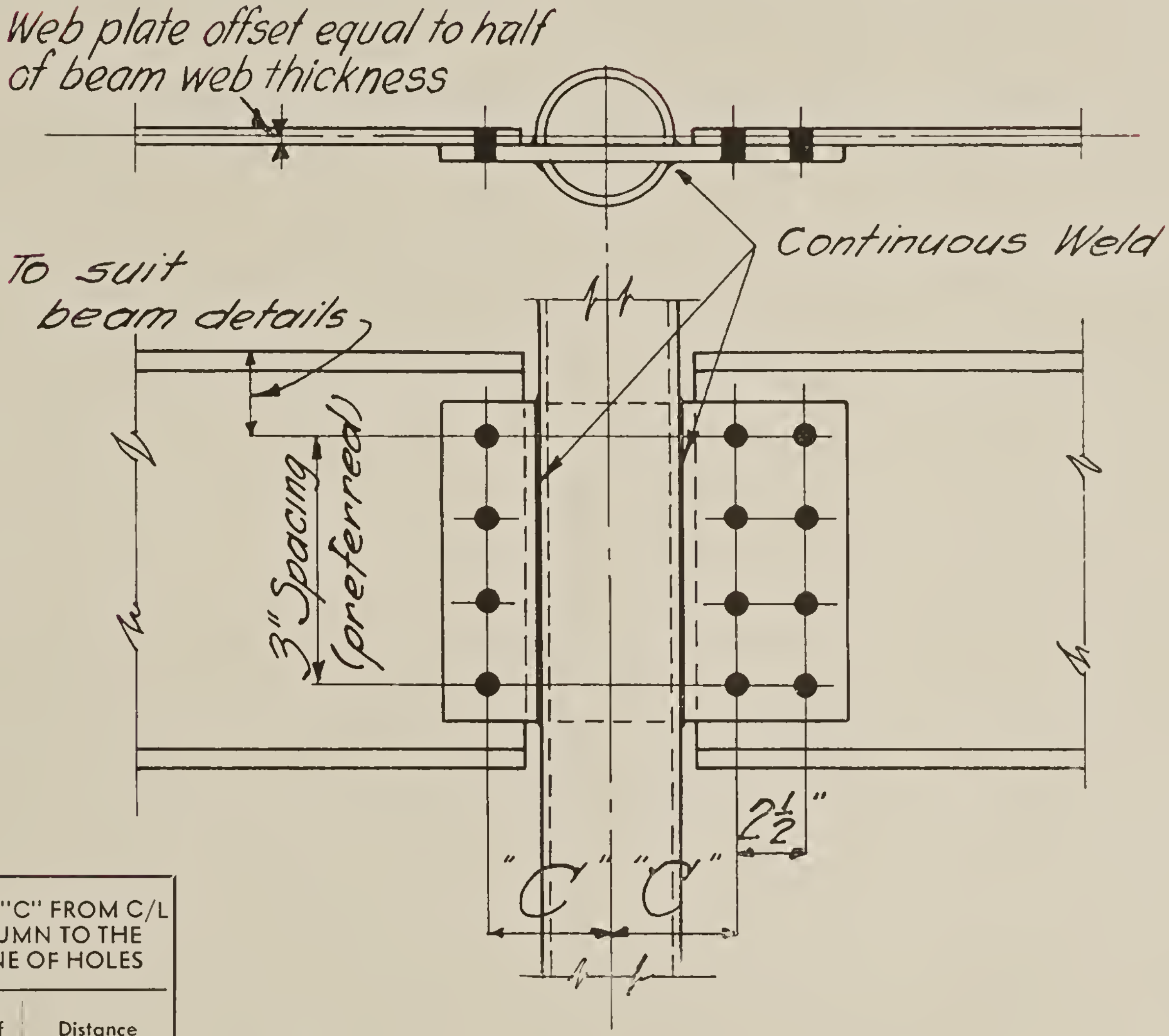


TABLE 9

DISTANCE "C" FROM C/L OF COLUMN TO THE FIRST LINE OF HOLES

Diameter of Column	Distance "C"
3½	3¾
4	4
4½	4¼
5	4½
5½	4¾
6⅝	5⅜
7⅝	5⅞
8⅝	6⅜
9⅝	6⅞
10¾	7⅜
12¾	8⅜

Safe loads for LALLY COLUMN WEB PLATE CONNECTIONS are given in Table 10.

Type "1" connections, using a single line of holes, will be found sufficient to carry ordinary loads. Type "2" connections, with a double line of holes, are used for heavier work.

Connections may be either field bolted or field riveted.

When a column carries a beam on one side only, or is otherwise eccentrically loaded, it must be checked for equivalent direct load as described in Tables 5 and 6.

LALLY COLUMN WEB PLATE CONNECTIONS

TABLE 10

SAFE LOADS

All loads are given in thousands of pounds.
For sketch and additional notes see preceding page.
For other allowable working stresses modify the Safe Loads proportionately.

Working Stresses — A.I.S.C. 1949			
Stress	on Rivets	on Bolts	on Web Pl.
Shear	15,000	10,000	13,000
Single Shear Bearing	32,000	20,000	......
Bending	......	......	20,000

BEAM or CHANNEL		TYPE 1—SINGLE LINE OF HOLES							TYPE 2—DOUBLE LINE OF HOLES						
		WEB PLATE		TOTAL NO. OF HOLES	SAFE LOAD				WEB PLATE		TOTAL NO. OF HOLES	SAFE LOADS			
					RIVETS		BOLTS					RIVETS		BOLTS	
Depth	Web	Depth	Thickness		¾"φ	⅞"φ	¾"φ	⅞"φ	Depth	Thickness		¾"φ	⅞"φ	¾"φ	⅞"φ
5, 6, 7	3/16								2½ to 5½	USE THICKNESS AS REQUIRED BY LOAD. SEE TABLE AT BOTTOM OF PAGE.	2	9.0	10.5	5.6	6.6
	¼											12.0	14.0	7.5	8.8
	5/16											13.3	17.5	8.8	10.9
	⅜											13.3	18.0	8.8	12.0
	7/16+											13.3	18.0	8.8	12.0
7, 8 9, 10	3/16	5½	½	2	9.0	10.5	5.6	6.6	5½ to 8½		4	18.0	21.0	11.2	13.1
	¼				12.0	14.0	7.5	8.8				24.0	28.0	15.0	17.5
	5/16				13.3	17.5	8.8	10.9				26.5	35.0	17.7	21.9
	⅜				13.3	18.0	8.8	12.0				26.5	36.1	17.7	24.0
	7/16+				13.3	18.0	8.8	12.0				26.5	36.1	17.7	24.0
12, 14	3/16	8½	½	3	13.5	15.8	8.4	9.8	8½ to 11½		6	27.0	31.5	16.9	19.7
	¼				18.0	21.0	11.3	13.1				36.0	42.0	22.5	26.3
	5/16				19.9	26.3	13.3	16.4				39.8	52.5	26.5	32.8
	⅜				19.9	27.1	13.3	18.0				39.8	54.1	26.5	36.1
	7/16+				19.9	27.1	13.3	18.0				39.8	54.1	26.5	36.1
15, 16, 18	5/16	11½	½	4	26.5	35.0	17.7	21.9	11½ Min.		8	53.0	70.0	35.4	43.8
	⅜				26.5	36.1	17.7	44.0				53.0	72.2	35.4	48.1
	7/16+				26.5	36.1	17.7	24.0				53.0	72.2	35.4	48.1
18, 20, 21	⅜	14½	½	5	33.2	45.1	22.1	30.1	14½ Min.	⅝	10	66.3	90.2	44.2	60.1
	7/16+				33.2	45.1	22.1	30.1				66.3	90.2	44.2	60.1
21, 24	⅜	17½	½	6	39.8	54.1	26.5	36.1	17½ Min.	⅝	12	79.6	108.2	53.0	72.1
	7/16+				39.8	54.1	26.5	36.1				79.6	108.2	53.0	72.1
24, 27	7/16+	20½	½	7	46.4	63.1	30.9	42.1	20½ Min.	⅝	14	92.8	126.3	61.9	84.1
27, 30	7/16+	23½	½	8	53.0	72.2	35.4	48.1	23½ Min.	⅝	16	106.1	144.3	70.7	96.2
30, 33	7/16+	26½	½	9	59.7	81.2	39.8	54.1	26½ Min.	⅝	18	119.3	162.4	79.6	108.2
33, 36	7/16+	29½	½	10	66.3	90.2	44.2	60.1	29½ Min.	⅝	20	132.6	180.4	88.4	120.2
36	7/16+	32½	½	11	72.9	99.2	48.6	66.1	32½ Min.	⅝	22	145.9	198.4	97.2	132.2

SAFE LOAD ON WEB PLATES IN KIPS (¾" RIVETS OR BOLTS)

Thickness of Plote	Depth of Plote—Inches																	
	2½	3	3½	4	5½	6	6½	7	8½	9	11½	12	14½	17½	20½	23½	29½	32½
½"	3.0	4.3	5.9	7.6	14.3	17.2	20.0	23.2	34.3	38.2	53.6	56.8	67.9	82.0	96.3	110.5	139.0	153.2
⅝"	3.8	5.4	7.3	9.5	17.8	21.4	25.0	29.0	42.8	47.8	67.0	71.0	84.9	102.5	120.4	138.1	173.8	194.1
¾"	4.5	6.4	8.9	11.4	21.5	25.8	30.0	34.8	51.5	57.3	80.4	85.2	101.9	123.0	144.8	165.8	208.5	230.0
⅞"	5.3	7.5	10.3	13.3	25.0	30.1	35.0	40.6	60.0	66.9	93.8	99.4	118.8	143.5	168.5	193.4	242.2	268.3

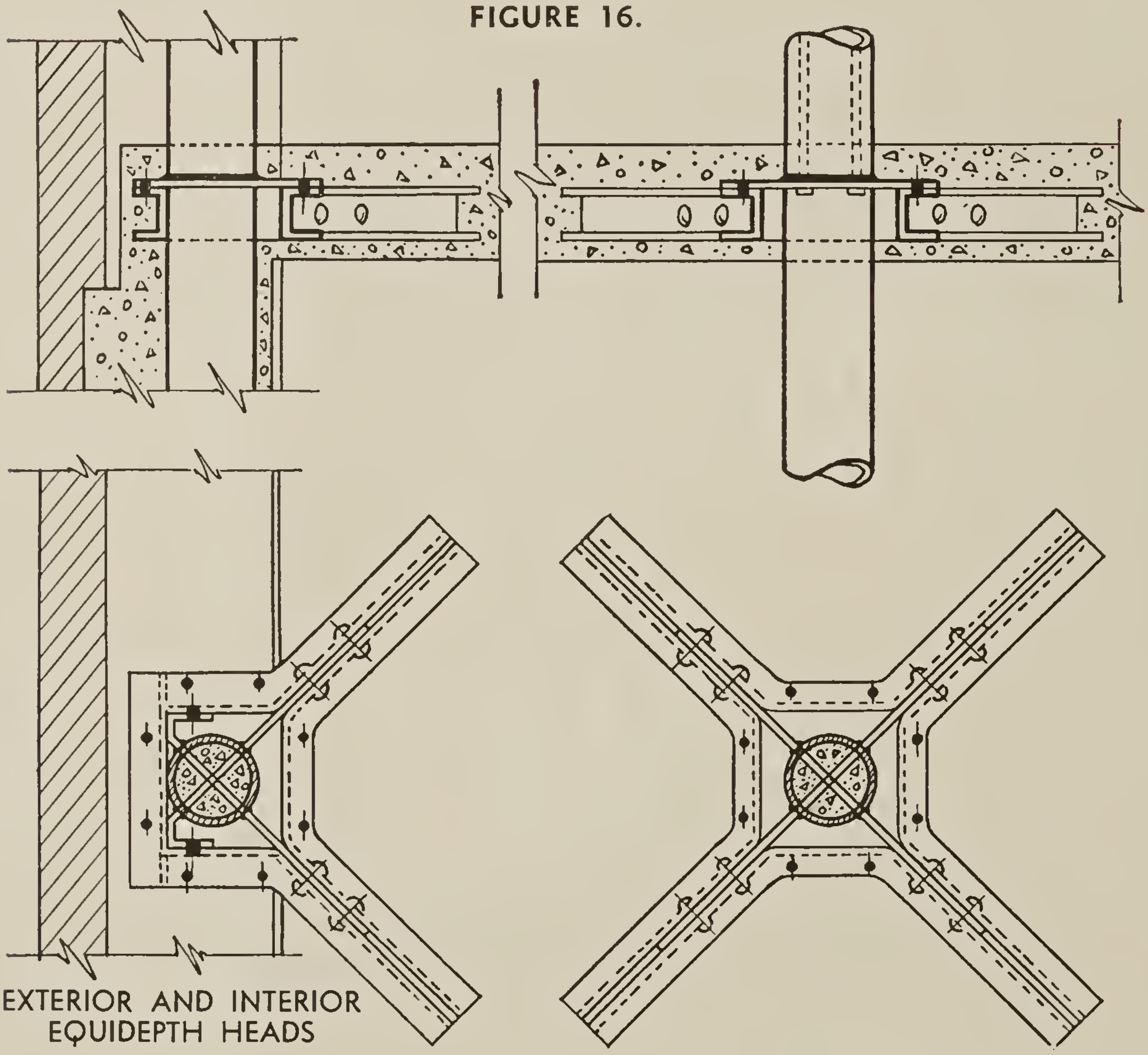
For Modern Design!

THE LALLY ORDER

LALLY EQUIDEPTH
REINFORCED CONCRETE CONSTRUCTION

THE Lally Column, originally described as a "New Type of Column" and more recently as contributing a new style to the art of building as the "SIXTH ORDER OF ARCHITECTURE," has now been adapted to use with the most economical form of reinforced concrete floor construction. In combination with the reinforced-concrete flat plate, the "LALLY ORDER" utilizes the advantages of the Lally Column with the pleasing and adaptable form of the round section, its uniform rigidity about all axes and the exceptional fire resistance of the concrete-filled pipe.

FIGURE 16.



EXTERIOR AND INTERIOR EQUIDEPTH HEADS

The new Lally construction adapts itself to the modern demand for maximum usable floor area, maximum sunlight and ventilation and minimum interference with architectural treatment. Columns and column constructions are prefabricated and delivered to the job with least delay. The Lally-Equidepth system combines the speed and ease of erection of the structural frame with the simplicity of form work plus adaptability and economy of the long span, reinforced concrete flat slab. It results in true flat ceilings without wet-plastering operations and with minimum maintenance and upkeep expense.

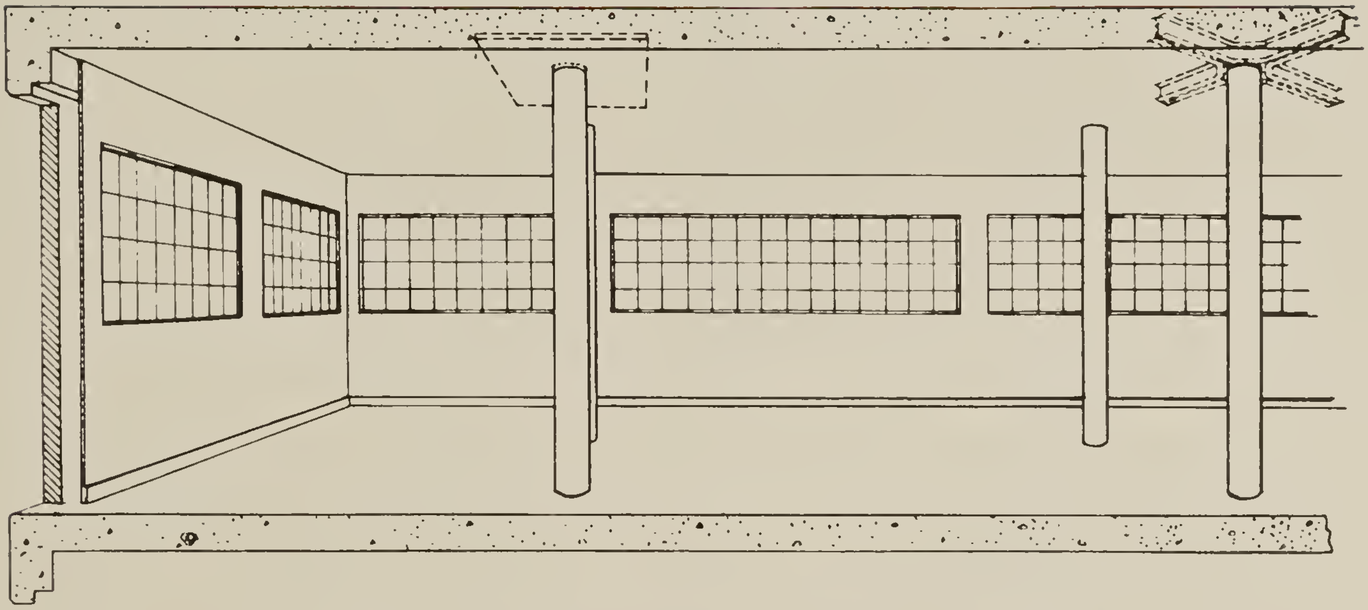
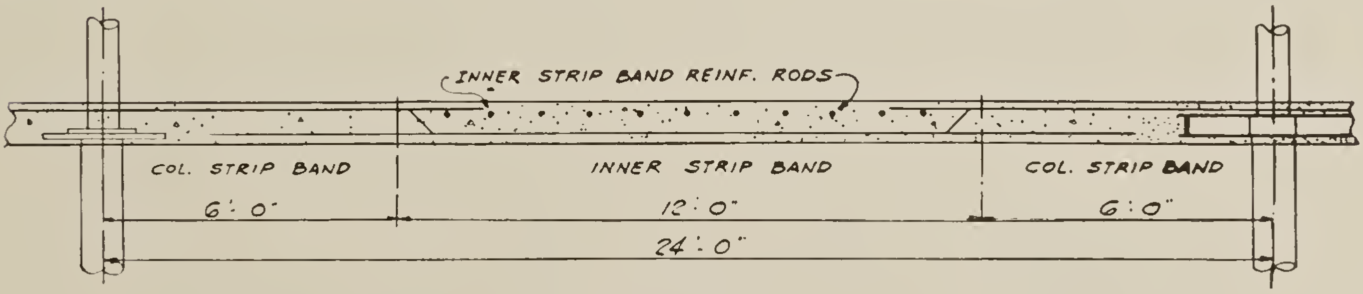
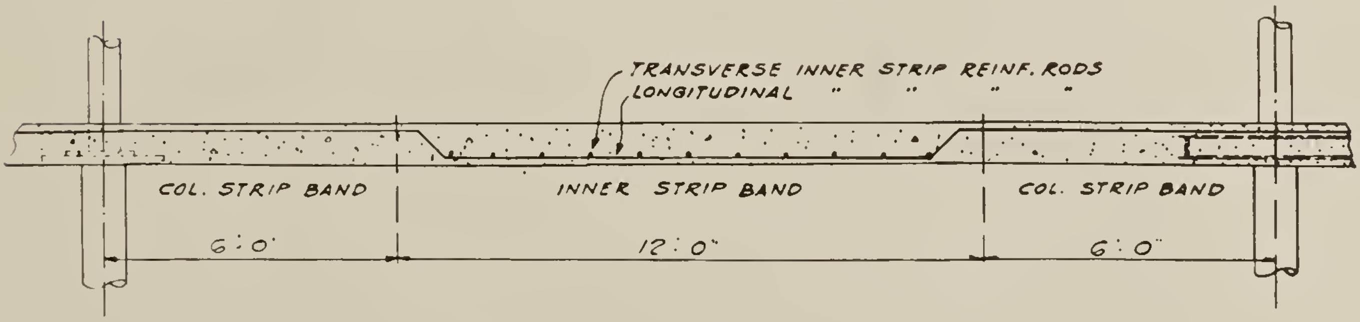


FIG 17. TYPICAL SECTIONS—LALLY EQUIDEPTH SYSTEM



COLUMN BAND



INNER BAND

TYPICAL EQUIDEPTH HEAD DETAILS

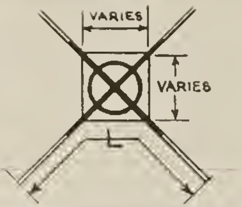
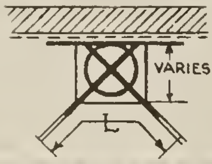
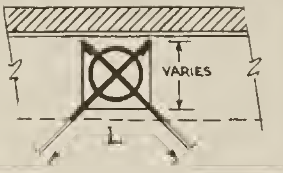
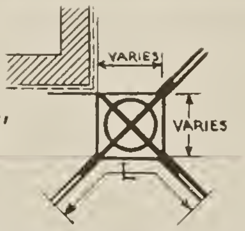
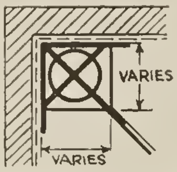
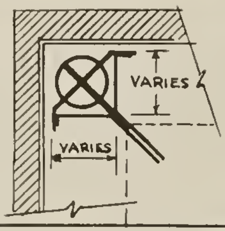
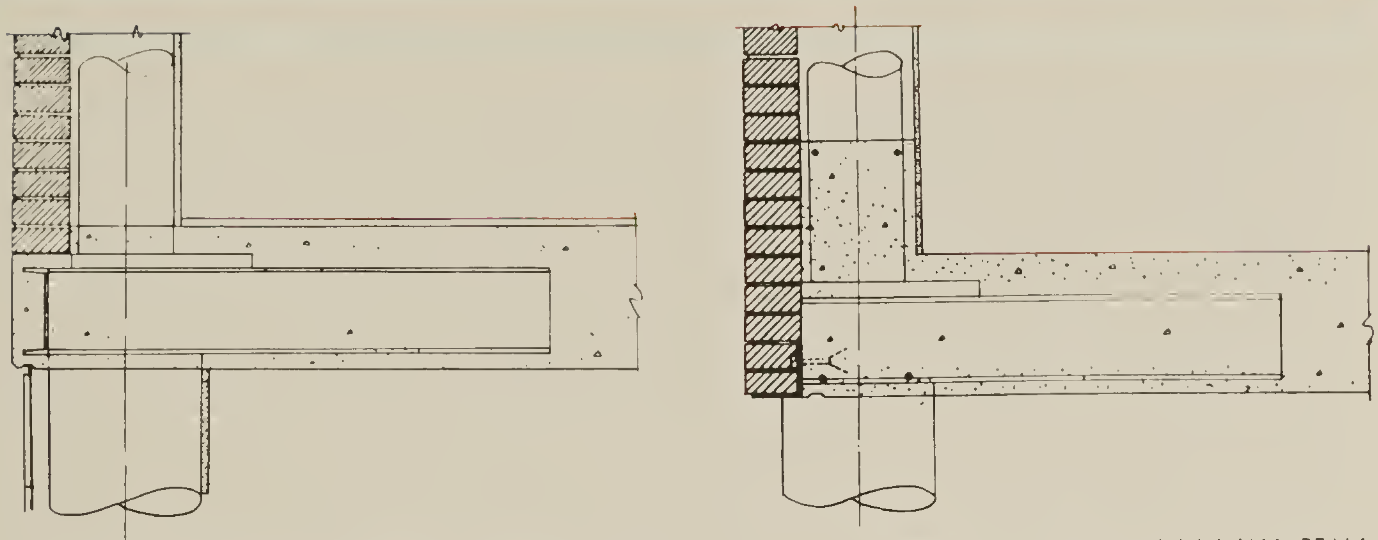
TYPE	LOCATION	REMARKS
"A" 	INTERIOR	For rectangular or circular columns
"B" 	WALL	Without wall lintels
"C" 	WALL	With wall lintels
"D" 	WALL (Right and Left)	Interior corners
"E" 	CORNER	Corner columns without wall lintels
"F" 	CORNER	Corner columns with wall lintels

TABLE II

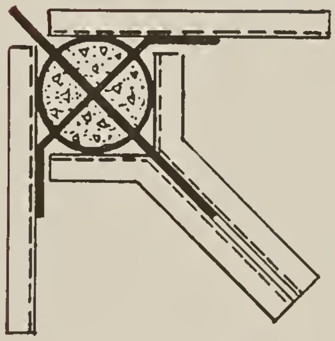


SPANDREL SECTION WITH EXPOSED CONCRETE BAND

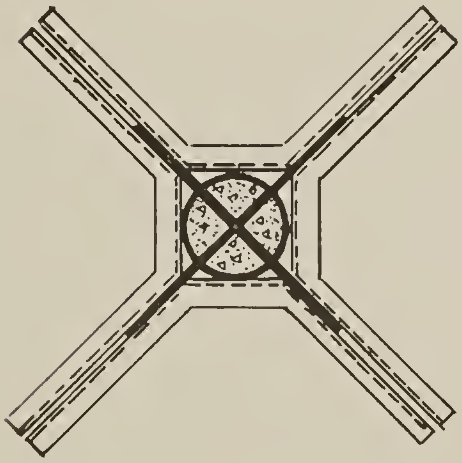
SPANDREL SECTION WITH WALL BEAM

FIGURE 18. SKELETON PANEL WALL CONSTRUCTION

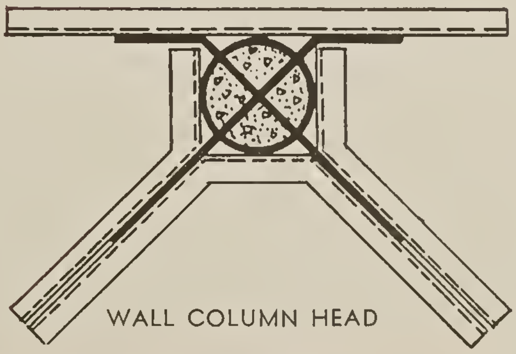
Fig. 18 illustrates two methods of adapting the channel-type head to skeleton panel-wall construction, both with and without a spandrel wall beam. The outer arms of the steel head can be omitted on the exterior columns when the spandrel wall beam is designed both to support the column band load and to resist the torsion incident to negative restraint of the floor band. The channel heads are furnished in the typical form shown in Table II and Fig. 19 for interior, wall and corner adaptations.



CORNER COLUMN HEAD



INTERIOR COLUMN HEAD



WALL COLUMN HEAD

FIGURE 19. TYPICAL CHANNEL HEADS

LALLY EQUIDEPTH CONSTRUCTION

DESCRIPTION

IN place of cumbersome ceiling and column capital projections, common in the customary flat slab construction, which require expensive formwork, the EQUIDEPTH SYSTEM substitutes a steel column-head or fixed-hinge of small steel shapes entirely concealed within the thickness of the floor plate as shown in Fig. 17. This form of construction was used in combination with reinforced concrete columns in the 1,500,000 square feet of the Williamsburgh Housing (Ten Eyck Houses) in Brooklyn, N. Y., and with structural steel columns in the 10-Story Museum of Modern Art in New York City, N. Y.

Under U. S. Patent 2,469,455, the Channel-Type head is now available with the concrete-filled, steel pipe LALLY COLUMN. An example of this use in residence construction is the 4-Story home for Mr. John W. Leopold, erected in Staten Island, N. Y., in 1950. Recently a 2-Story and basement office building was completed at Morristown, N. J., by the William H. Blanchard Company of Newark, N. J., for the Hooper Holmes Insurance Bureau (Fig. 20). Other installations in New Jersey, either completed or in course of construction, include the Somerset Hills National Bank at Basking Ridge, the Nursery and Training School at Totowa and the J. & S. Tool Co. Offices at Livingston.

The Leopold Residence shown in the photographs is unique in appearance, location and design. It is situated some 500 feet above sea level on Emerson Hill with unobstructed view of New York Harbor and Lower Bay. It is enclosed with a large expanse of glass area and is oriented so as to secure the maximum benefit of solar heat. The building contains a volume of 40,000 cubic feet and is provided with a system of automatically controlled radiant heating in the floor slabs, which are 7 inches thick with a 2 inch top finish of lightweight concrete. Experience during the past winter indicates a total consumption of 4,000 gallons of fuel oil at a cost of $400 for the heating season, during which the structure was subjected to unusual winds as high as 100 miles per hour velocity on two occasions (Fig. 21).

PATENTS PENDING

In addition to the Channel Type hinge, the LALLY COLUMN HEAD is also furnished in several forms of shop-welded plates illustrated in Figs. 22 and 23 of the "BASKET," "STIRRUP," "LOOP" and "STUD" types covered by U. S. Patent applications now pending.

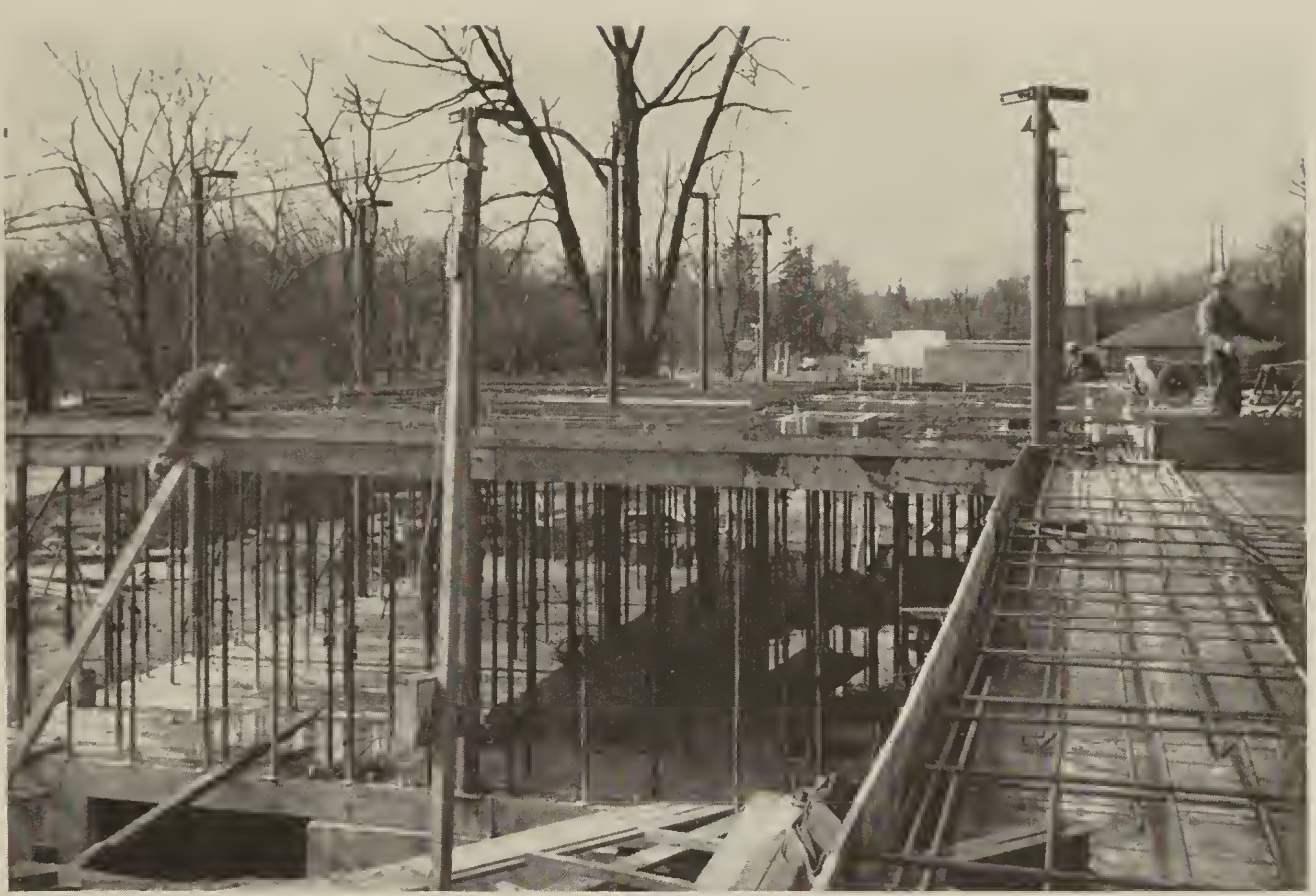
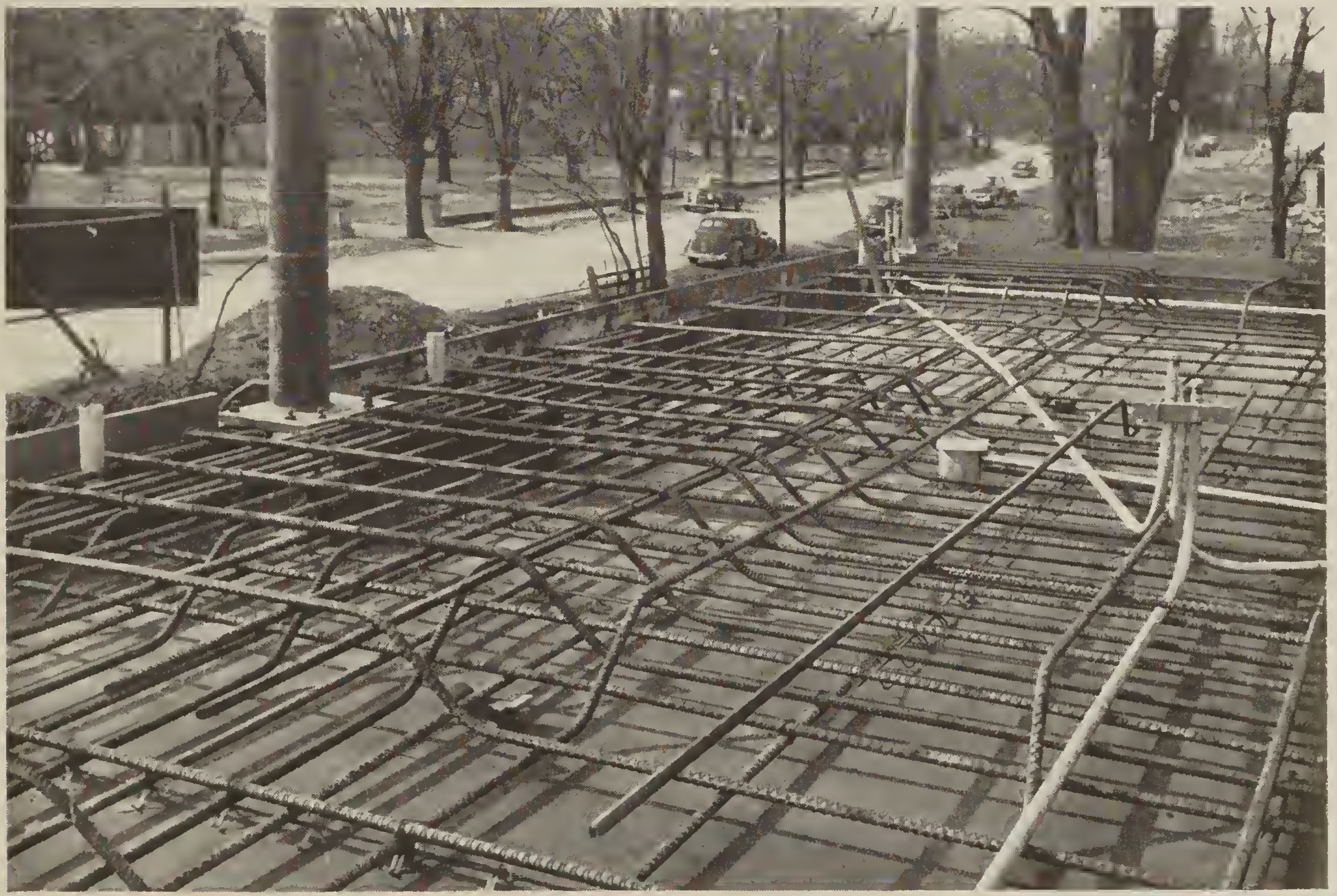


FIGURE 20. HOOPER HOLMES INSURANCE BUILDING, MORRISTOWN, N. J.
EDWIN KLINE, Architect



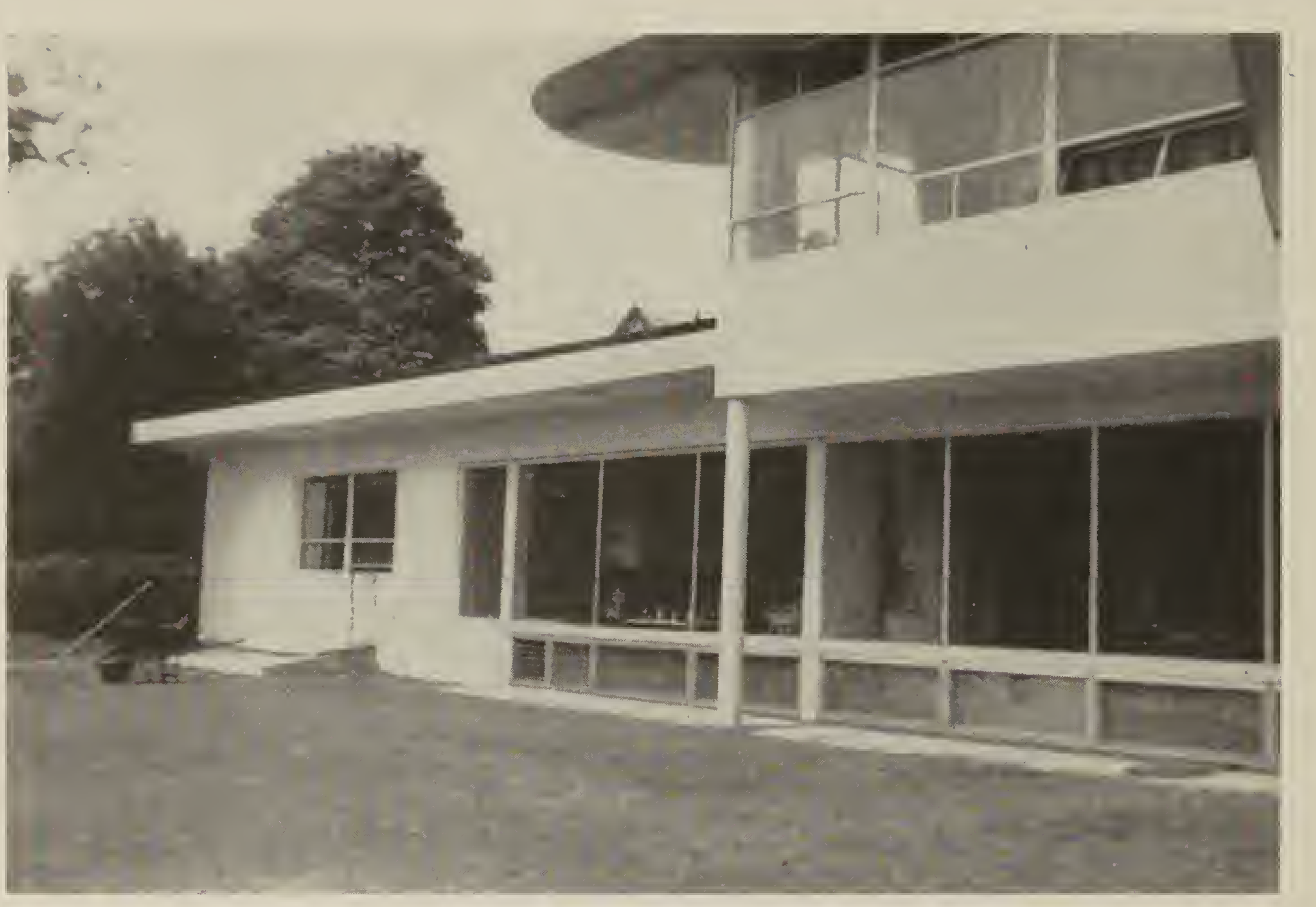
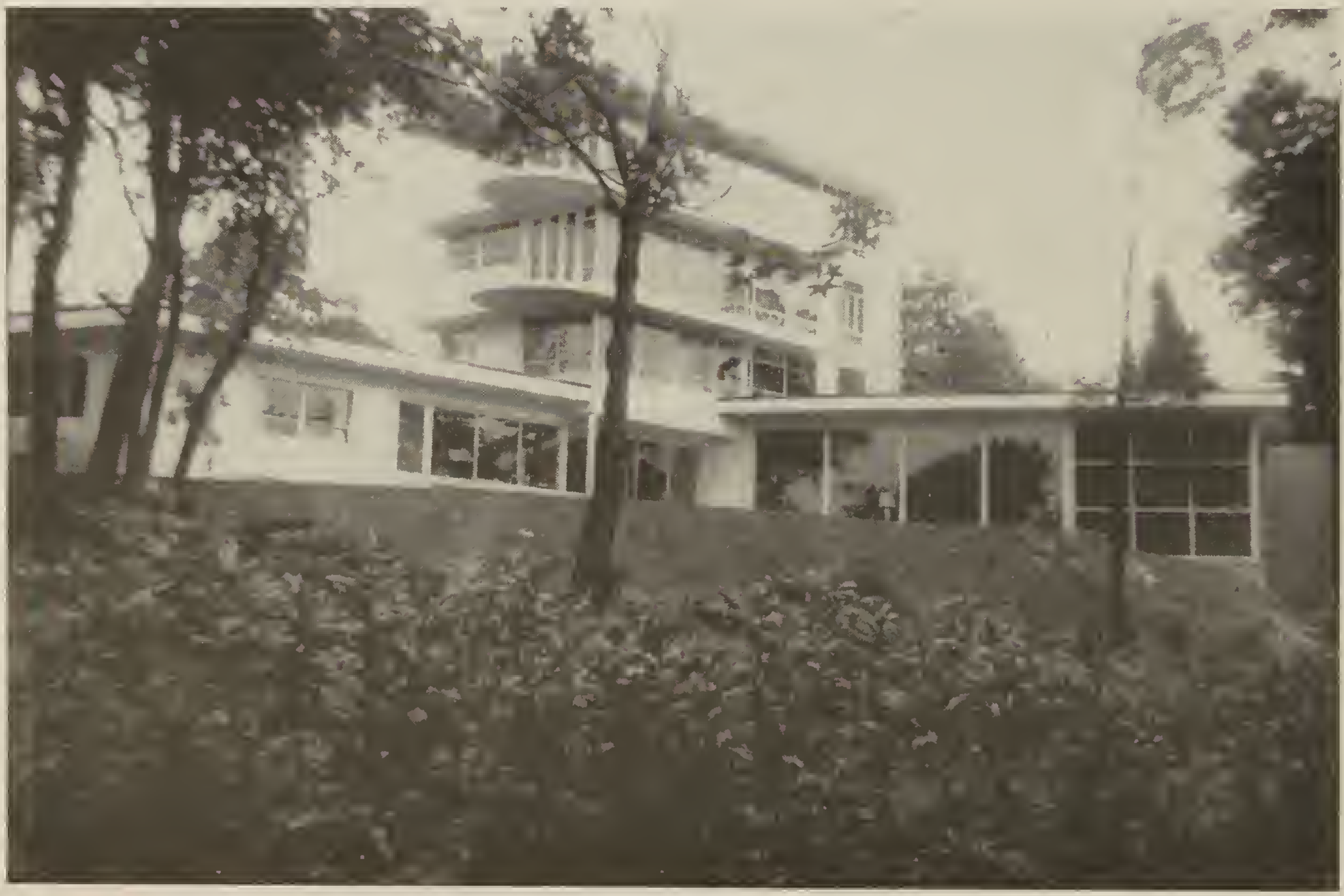
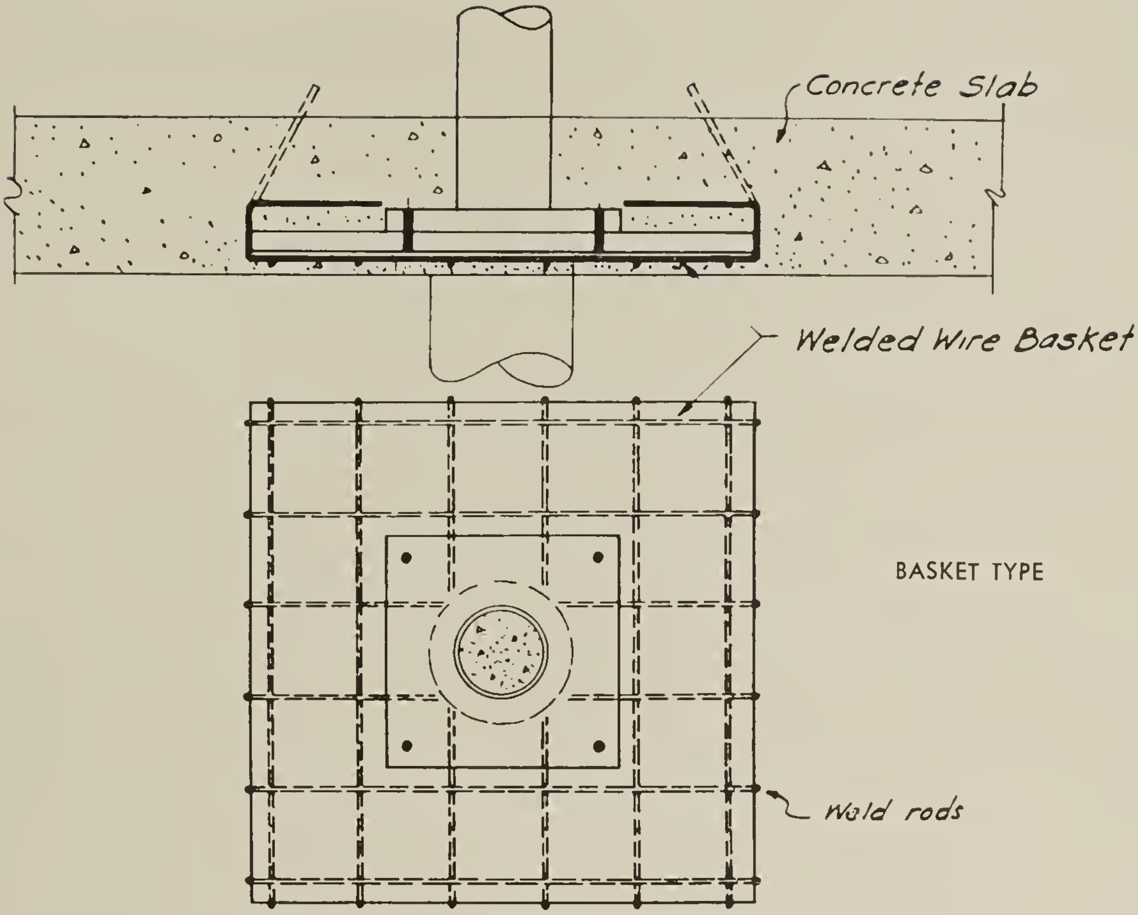


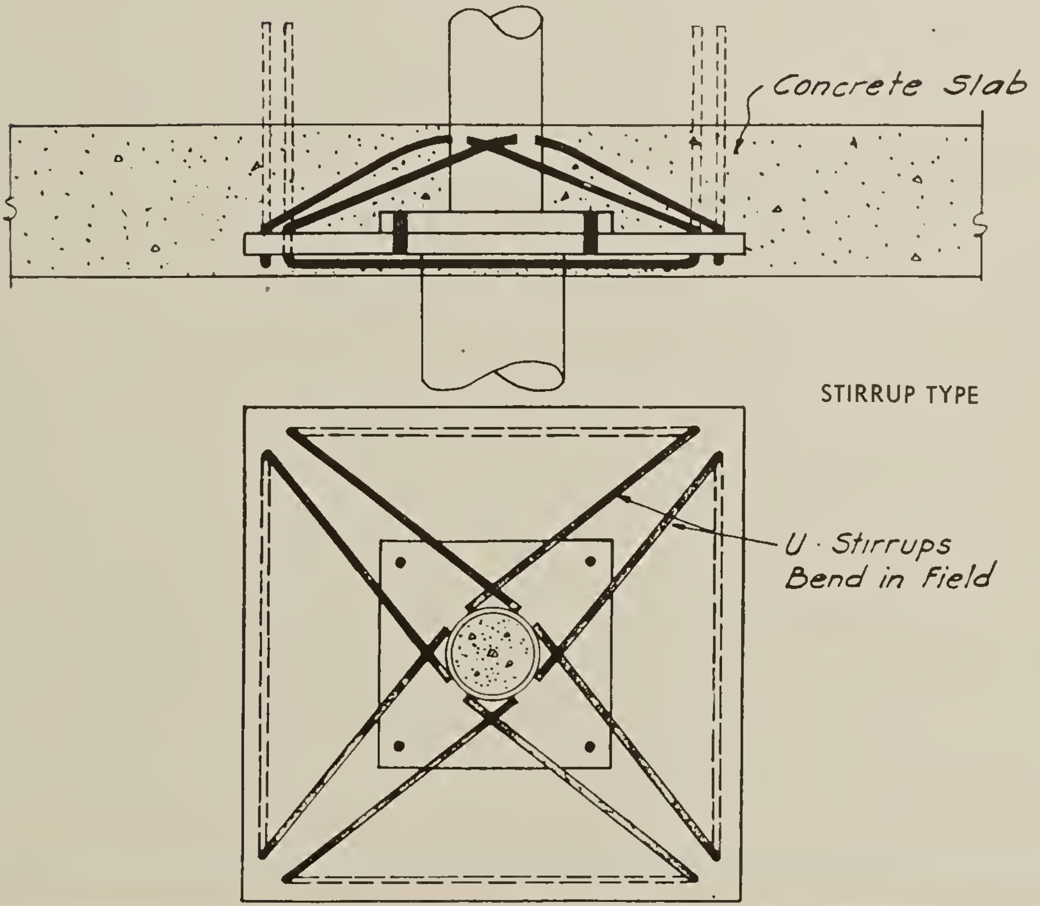
FIGURE 21. LEOPOLD RESIDENCE, EMERSON HILLS, S. I.
MATTHEW R. LEIZER, Architect



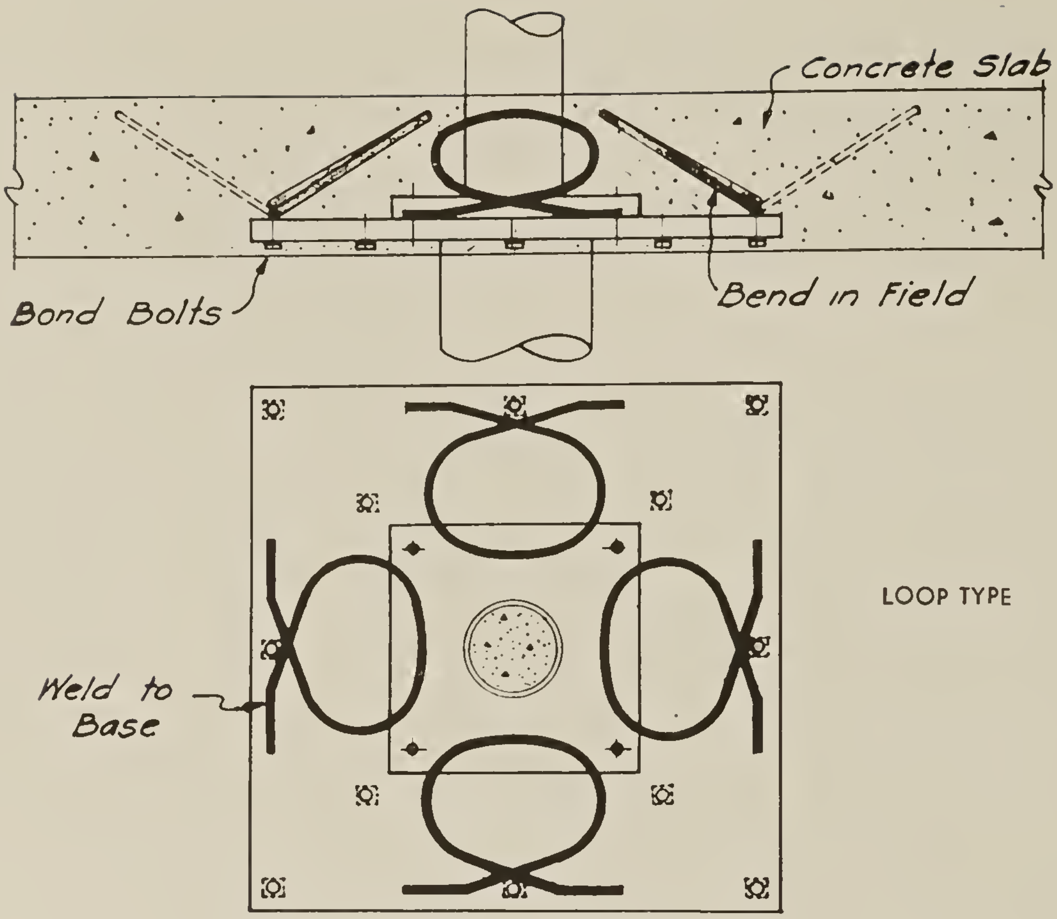


BASKET TYPE

FIG. 22. LALLY PLATE HINGES

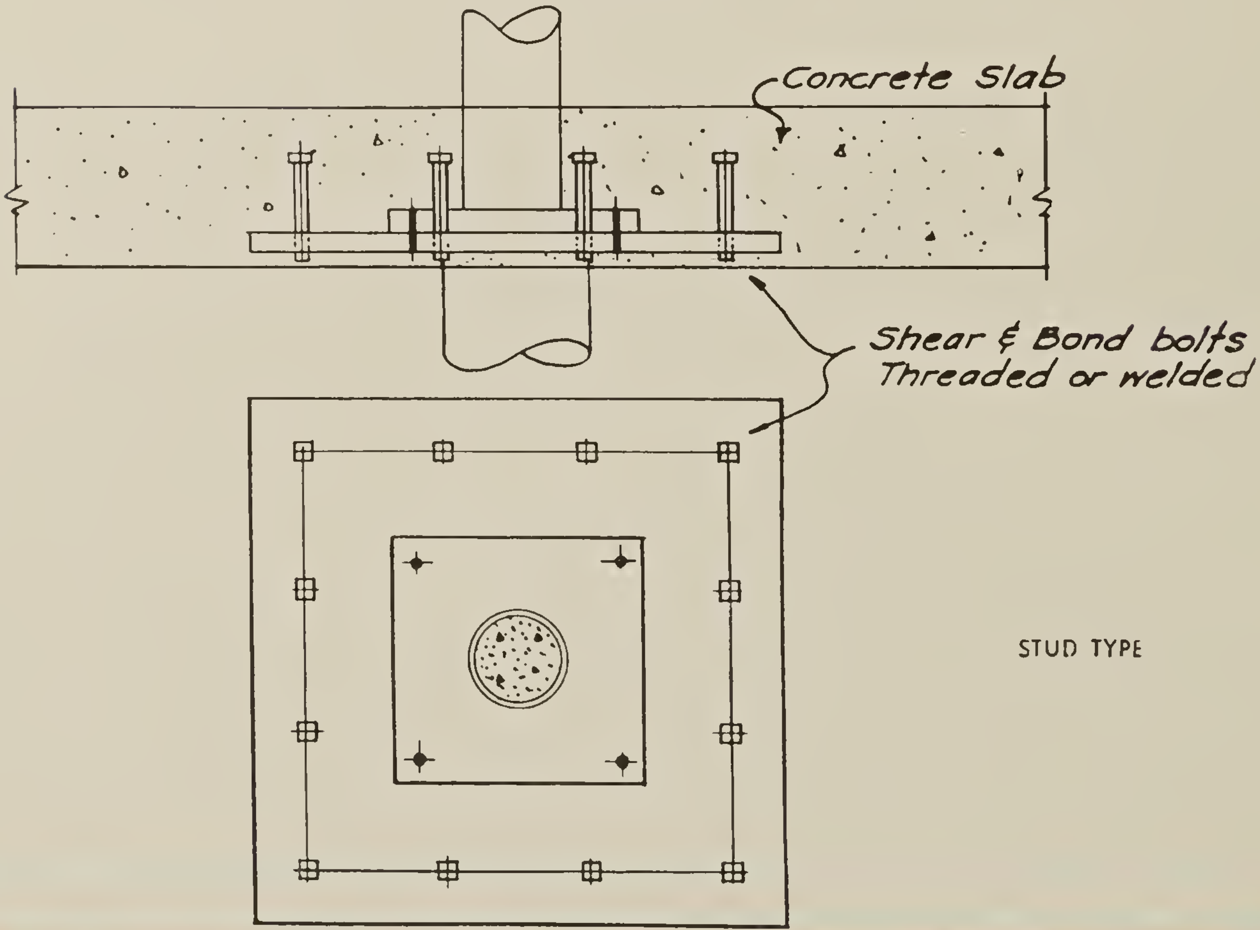


STIRRUP TYPE



LOOP TYPE

FIG. 23. LALLY PLATE HINGES



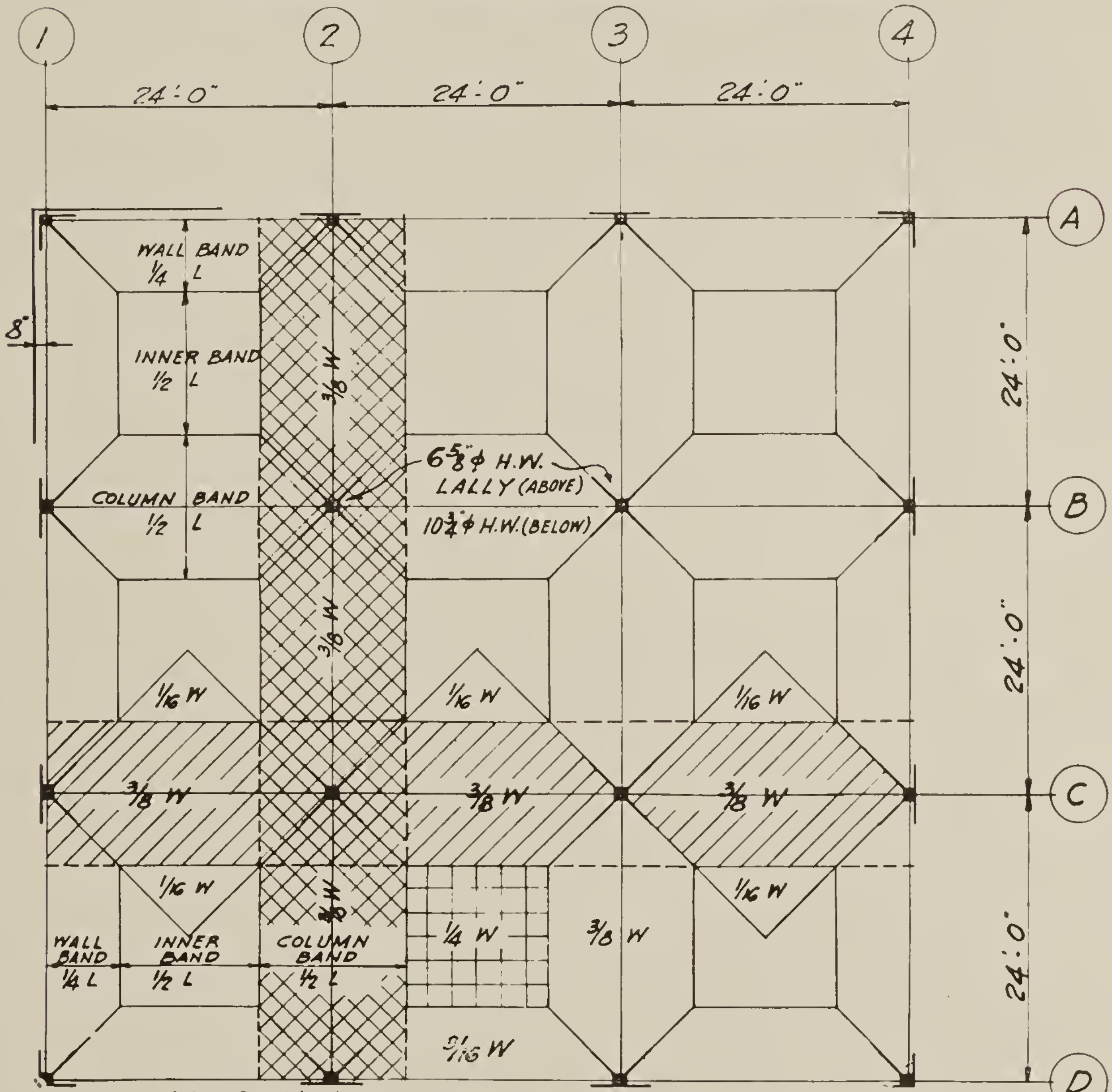
STUD TYPE

THE EQUIDEPTH FRAME DESIGN

FRAME DESIGN ANALYSIS

To illustrate the rigid frame analysis in the design of the LALLY EQUIDEPTH FRAME, a typical floor plan of a building with three equal bays, longitudinally and transversely, is shown in Fig. 24 for an assumed live load of 75 lbs. per sq. ft. The arrangement of Main Column Bands and Inner Suspended Bands, together with the method of load distribution is illustrated diagrammatically. Each of the two (2) Main Column bands directly supports three-eighths (3/8) of the full panel load. The two (2) inner bands support the remaining

FIGURE 24. LOAD DISTRIBUTION DIAGRAM



W = Panel load.
Load on suspended panel distributed inversely to ratio of cube of spans.
See Fig. 25 for design of band C (1-2-3-4). Inner bands designed as two-way continuous slabs. Width of band equals one half panel length.

one-quarter (¼) of the panel load, which in turn is transmitted one-eighth to each main band. If the panel is rectangular in shape, the inner band loads are distributed in proportion to the inverse ratio of the cube of the respective spans.

COLUMN DESIGN ANALYSIS

The calculation of moments for a typical Column Band B (1-2-3-4) is shown in Fig. 25. Although graphical delineation apparently indicates a triangular load on each main band, the actual load approaches a combination of uniform and parabolic distribution. This is further modified by reason of plate action, the stiffer corners of the panel supporting a greater proportion of the indicated panel load. In these computations the assumed fixed-end moment is the average ordinate of the moment area of the uniformly loaded beam or 1/12 W′.L., in which W′ is the load on the band in question. If W represents the total panel load, the fixed-end moment assumed in the computation is expressed as 1/24 W.L.

The quantities "A" and "C" are the resultant total end moments of the band in the rigid frame with alternate panels loaded and the sum of "A" and "C" gives the computed end moments with all panels loaded. The quantities "B" and "D" represent the end band moments under dead load only for the same condition of alternate panel loads. The sum of "A" and "D" and the sum of "B" and "C" therefor give the moments, respectively, for both unbalanced load conditions.

While the design herein indicated calls for continuous bent rods, modern procedure in the construction of flat slabs tends toward the use of straight bars for both top negative reinforcement over the column head and bottom positive reinforcement at the center of the bands.

COLUMN HEAD DESIGN

The moment resisted by the columns at each panel joint is the difference between the adjacent band moments. In this analysis, the columns are designed for the maximum unbalanced condition. If the columns are designed only for the unbalanced moment under full uniform panel loading, the design moments would be reduced on interior and exterior columns. Under customary building code requirements for uniform loading, a saving could be effected in the design of buildings in use groups that insure uniform live loads. The effect of unloaded alternate floors was considered an unnecessary refinement in the column design.

The total column moment at any panel joint is resisted by the upper and lower columns in the ratio of their relative stiffness factors. These stiffness factors are given in Fig. 26. The section properties and resisting moments of the LALLY COLUMN are obtained from Tables 5 and 6. The Column Schedule in Fig. 26 tabulates the resultant of direct and equivalent direct loads corresponding to the bending moment for a typical interior and exterior column and also gives the design of the column heads for bending.

LALLY PLATE HINGE DESIGN

The LALLY EQUIDEPTH FIXED HINGE is designed to transmit to the columns the portion of the moment of the rigid frame based on the relative rigidity of the columns and intersecting bands at the panel joint. The cross-sectional area of the steel head must be sufficient to resist the total end shear or direct column load on eight (8) channel sections in the case of a typical interior column. In addition, the arms of the channel-type or the extension of the welded-plate types must be of sufficient length to reduce the shear in the floor plate on the effective depth of the slab to a unit stress of 3 per cent of the compressive strength of the concrete. The channels in the problem herein discussed must be extended to provide a square of 55 by 55 inches in dimension, or approximately one-fifth (1/5) of the panel length.

In this example, the concrete is assumed with an ultimate compressive strength of 2,500 lbs. sq. in., the reinforcement as Intermediate Grade Billet Steel with a working stress of 20,000 lbs. sq. in. and the structural steel column-heads are designed with a fiber stress of 24,000 lbs. sq. in. This higher stress is justified as the structural shapes are completely embedded in and act as a composite unit with the concrete.

LALLY EQUIDEPTH HEADS may be fabricated by either welding, riveting or bolting. In any one band the bending resistance of the channels is assumed to be provided by the projected section of four (4) channels in each band at right angles to the span length. The resisting section in bending therefore consists of two and eight-tenths (2.8) channels at both ends of the band at interior columns and one and four-tenths (1.4) channels at corner columns in a square panel. Table 12 lists the approximate design of channel equidepth heads for live loads varying from 60 to 500 lbs. per sq. ft. for a typical interior square panel. As the moment resisted by the column and consequently the size of the column head will depend on the size of the column which in turn will vary with the height of the building, these values should be used for estimate only.

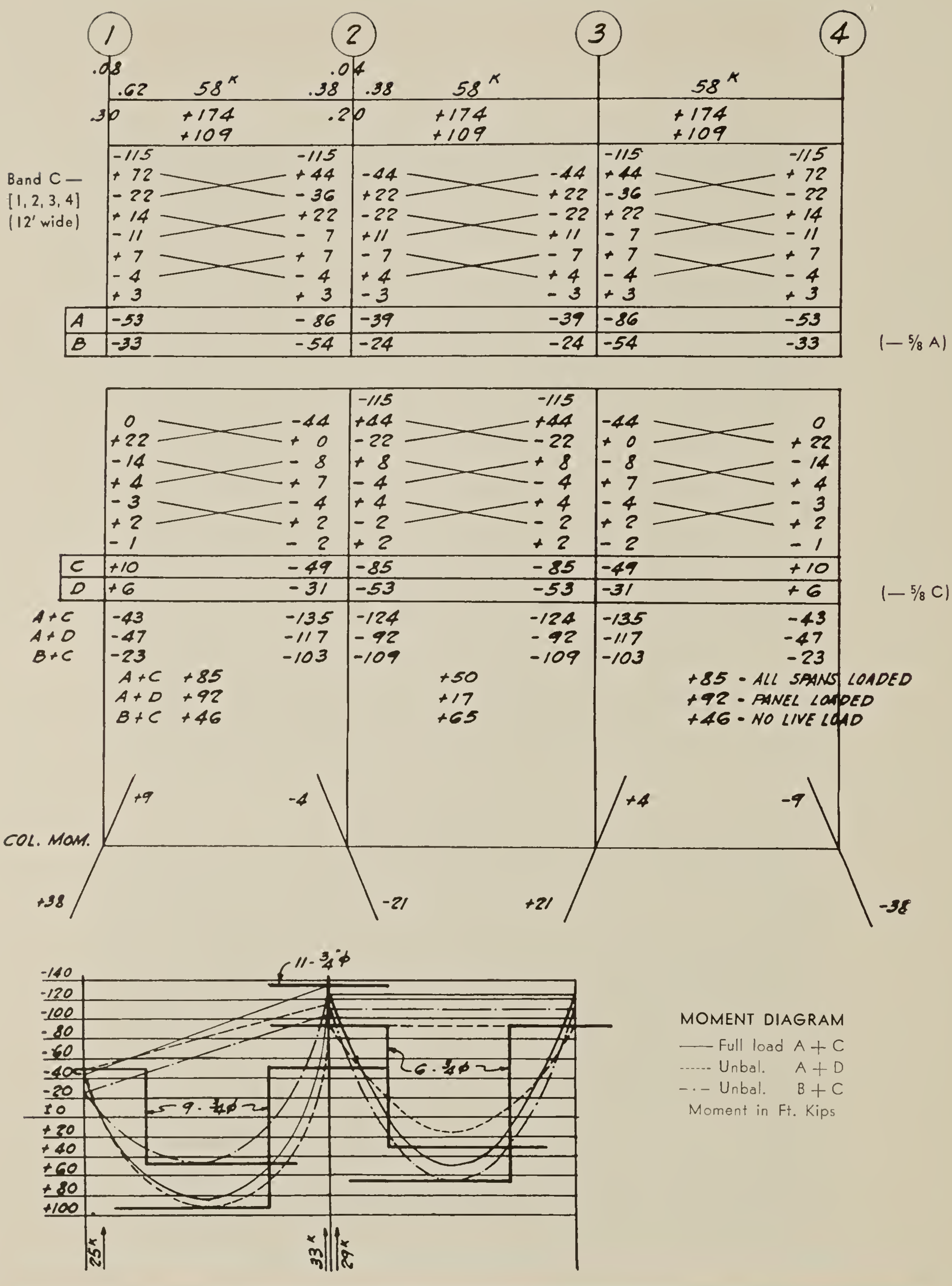
The design of the LALLY PLATE HINGE follows the same procedure as that described for the Channel Type Heads.

The lateral dimensions of the plate are such as to reduce the shear in the slab to 3 per cent of the ultimate compressive strength of the concrete.

The cross-sectional area of the plate is designed to transmit the total panel load to the column in shear.

The section modulus of the plate is adequate to transmit that portion of the total negative moment resisted by the columns in proportion to the relative rigidity factors of the columns and slab as described under the column design analysis. At intermediate floor levels the total column moment at any panel point is resisted by the upper and lower columns in the ratio of their relative stiffness factors while the section modulus of the plate is designed in all cases for the total column moment, or the difference in the end moments of adjoining column bands.

The stirrups or bond rods attached to the LALLY PLATE HEAD serve dual purposes. The projections below the plate are ties to prevent cracking or spalling of the concrete covering. The total cross-sectional area of the stirrups or rods above the plate is adequate to



	-115	-115			-115	-115
	+72	+44	-44	-44	+44	+72
	-22	-36	+22	+22	-36	-22
	+14	+22	-22	-22	+22	+14
	-11	-7	+11	+11	-7	-11
	+7	+7	-7	-7	+7	+7
	-4	-4	+4	+4	-4	-4
	+3	+3	-3	-3	+3	+3
A	-53	-86	-39	-39	-86	-53
B	-33	-54	-24	-24	-54	-33

			-115	-115		
	0	-44	+44	+44	-44	0
	+22	+0	-22	-22	+0	+22
	-14	-8	+8	+8	-8	-14
	+4	+7	-4	-4	+7	+4
	-3	-4	+4	+4	-4	-3
	+2	+2	-2	-2	+2	+2
	-1	-2	+2	+2	-2	-1
C	+10	-49	-85	-85	-49	+10
D	+6	-31	-53	-53	-31	+6
A+C	-43	-135	-124	-124	-135	-43
A+D	-47	-117	-92	-92	-117	-47
B+C	-23	-103	-109	-109	-103	-23

A+C +85
A+D +92
B+C +46

+50
+17
+65

+85 - ALL SPANS LOADED
+92 - PANEL LOADED
+46 - NO LIVE LOAD

FIG. 25. COLUMN BAND MOMENT CALCULATION

COLUMN AND SLAB FACTORS

10″ Slab — 2500# Concrete

SLAB FACTORS

I (Band per ft.) $= 1/12 \times 12 \times 10^3 = 1000$

I/L " " " $= 1000 \div 24 = 42$

I/L ($6\frac{5}{8}\ \phi$ Col.) $= 12 \times 35.25 \div 9.8 = 43$

I/L ($10\frac{3}{4}\ \phi$ Col.) $= 12 \times 207.75 \div 9.8 = 255$

I/L ($7\frac{5}{8}\ \phi$ Col.) $= 12 \times 58.48 \div 9.8 = 70$

Distribution of Moments

Hinge ① & ④

$7\frac{5}{8}\ \phi$ Col. — 70 = .08
$10\frac{3}{4}\ \phi$ Col. — 255 = .30
12′ Slab — 504 = .62
829 1.00

Loads	D.L.	L.L.	T.L.
Roof	110	40	150
2nd Flr.	125	75	200

Ratio of Dead To Total Load $X = \frac{125}{200} = \frac{5}{8}$

"w" per panel $= 115^{\mathrm{K}}$

Hinge ② & ③

$7\frac{5}{8}\ \phi$ Col. — 70 = .04
$10\frac{3}{4}\ \phi$ Col. — 255 = .20
12′ Slab — 504 = .38
12′ Slab — 504 = .38
1333 1.00

Fixed end mom. uniform load $= 1/12 \times 24 \times \dfrac{200 \times 576}{2} = 115^{1\mathrm{K}}$

Resisting Moments

1 ft. 10″ Slab $= 12^{1\mathrm{K}}$
1 □″ Reinf. $= 12^{1\mathrm{K}}$
$\frac{3}{4}\ \phi$ $= 5.2^{1\mathrm{K}}$

Reactions

$R_1 \times 24 - 58 \times 12 - 43 = -135$
$R_1 = 25^{\mathrm{K}}$
$R_2 = 33 + 29 = 62\mathrm{K}$

Band moments calculated for $7\frac{5}{8}''\ \phi$ upper column rechecked for $6\frac{5}{8}''\ \phi$, which is found adequate. In any case, when the initial calculation shows the assumed column sizes to be either too large or too small, the analysis must be repeated for the new column sizes.

COLUMN SCHEDULE

COL. No.	C1 (1)	C1 (2)	C1 (3)	C1 (4)	C2 (1)	C2 (2)	C2 (3)	C2 (4)
	DIR. LD K	M 'K	EQUIV. K	TOTAL DIR K	DIR. LD. K	M 'K	EQUIV. DIR. K	TOTAL DIR K
ROOF (2nd Story col.)	36	9	80	116 $6\frac{5}{8}\phi$	96	4	36	132 $6\frac{5}{8}\phi$
2nd FLOOR (1st Story Col.)	82	38	230	312 $10\frac{3}{4}\phi$	224	21	119	343 $10\frac{3}{4}\phi$

Each column is designed for sum of direct loads on all tiers above plus bending at tier in question shown in column 4 of schedule; or the load in column 1 plus the load in column 3 of the schedule. Moment of Inertia in terms of the equivalent section are determined by multiplying the steel equivalent from the "Lally Table of Eccentric Loads." This table also gives the equivalent direct load for each $10''^{\mathrm{K}}$ of bending moment. See Lally Handbook.

COLUMN HEADS

Exterior Columns: S.M. $= \dfrac{47 \times 12}{24 \times 2.8} = 8.7$ use 8[11.5 or 6[16.3

Interior Columns: S.M. $= \dfrac{25 \times 12}{24 \times 2.8} = 4.5$ use 6[8.2 or 4[13.8

The interior heads are made up of four (4) chnanels 6′-0″ long and the exterior heads of one (1) channel 6′-0″ long, one (1) channel 4′6″ long and two (2) channels 3′-6″ long as shown in Fig. 19.

FIG. 26. COLUMN AND COLUMN HEAD DESIGN

LALLY EQUIDEPTH HEAD SECTIONS

FOR VARYING SLAB THICKNESS

Span Feet	Slab Inches	LIVE LOADS LBS. SQ. FT.										Thick-ness SLAB
		60*	70*	75	80*	100	120	150	200	300	500	
14	6″	3[4.1 2′-4″	3[4.1 2′-4″	3[4.1 2′-6″	3[4.1 2′-6″	3[4.1 3′-0″	3[4.1 3′-6″	4[5.4 3′-6″	4[5.4 4′-0″	4[5.4 4′-6″	5[9 7′-0″	8″
15		2′-6″	2′-6″	2′-9″	3′-0″	3′-0″	4[5.4 3′-6″	3′-9″	4[5.4 4′-0″	5′-6″	6[10.5 8′-0″	9″
16		3′-0″	3′-6″	3′-0″	3′-0″	4[5.4 3′-6″	3′-6″	4[5.4 3′-6″	4′-6″	5[6.7 6′-0″	8′-6″	
17		3′-6″	3′-6″	3′-6″	4[5.4 3′-6″	4′-0″	4[5.4 3′-9″	4′-0″	5[6.7 5′-0″	6′-0″	6[12 8′-6″	10″
18		3′-6″	4′-0″	4′-0″	4′-0″	4[5.4 4′-0″	4′-0″	5[6.7 4′-6″	5′-0″	6[8.2 6′-6″	6[15.3 9′-6″	
19		4′-0″	4′-6″	4′-6″	4′-6″	4′-6″	5[6.7 4′-6″	5′-0″	6[8.2 5′-6″	7′-6″	6[18 10′-0″	11″
20		4′-6″	5′-0″	4[5.4 4′-6″	4′-6″	5′-0″	5′-0″	6[8.2 5′-6″	6′-0″	6[8.2 7′-6″	7[20.35 10′-6″	
21		5′-0″	4[5.4 5′-0″	5′-0″	4[5.4 4′-9″	5′-6″	5′-6″	5′-9″	6[8.2 6′-6″	6[12 8′-0″	8[22.7 11′-0″	12″
22	7″	4[5.4 5′-0″	5′-6″	5′-6″	5[6.7 5′-0″	5[6.7 5′-6″	6′-0″	6′-0″	7′-0″	8[11.5 8′-6″	8[28.2 12′-0″	
23		5′-6″	6′-0″	6′-0″	5′-6″	6′-0″	6[8.2 6′-0″	7′-0″	8′-0″	9′-6″	12[20.7 11′-6″	16″
24		4[7.25 6′-0″	5[6.7 6′-0″	5[6.7 6′-6″	6′-0″	6[8.2 6′-6″	6′-6″	6[8.2 7′-0″	6[10.5 8′-6″	8[18.75 10′-0″	12′-6″	
25	8″	5[6.7 6′-6″	6′-6″	6′-9″	6′-6″	7′-0″	7′-0″	8′-0″	8[11.5 8′-6″	8[20 11′-0″	12[35 13′-6″	
26		6′-9″	5[9 7′-0″	5[9 7′-0″	6[8.2 7′-0″	6[10.5 7′-6″	6[10.5 8′-0″	6[12 8′-6″	9′-0″	12[20.7 10′-0″	12[35 14′-0″	
27		5[9 7′-0″	6[8.2 7′-6″	6[8.2 7′-6″	7′-6″	6[12 8′-0″	6[12 8′-6″	6[15.1 9′-0″	8[18.7S 10′-0″	11′-0″	12[46.6 15′-0″	
28	9″	6[8.2 7′-6″	6[10.5 8′-0″	6[10.5 8′-0″	6[10.5 8′-0″	8′-6″	6[12 9′-0″	6[18 9′-6″	10′-6″	12′-0″	12[50 16′-0″	
29		6[10.5 8′-0″	6[12 8′-6″	6[10.5 8′-0″	6[12 8′-6″	6[15.1 9′-0″	6[15.1 9′-6″	8[16.25 10′-0″	8[21.4 11′-6″	12[30 13′-0″		
30		6[12 8′-6″	6[12 8′-6″	6[12 8′-6″	9′-0″	6[15.1 9′-6″	6[18 10′-0″	8[18.7S 10′-6″	12[20.7 11′-0″	12[35 14′-0″		
SLAB THICK			10″			11″		12″	16″			

TABLE 12

*Includes 20# partitions.

The values for slab thickness and column heads in Table 12 apply only to average conditions for interior square panels and are to be used for estimate purposes only. A complete rigid frame analysis should be made for each specific building as described on pages 40 to 46, inclusive.

resist the horizontal shear in the slab in both directions thus insuring the fixity of the hinge and preventing any differential movement between column head and slab. This movement is prevented in the case of the CHANNEL-TYPE HEAD by the complete embedment and interlocking of the channel sections in the concrete slab.

Let $W =$ total panel load in lbs.
$M =$ total negative moment of a column band in ft.-lbs.
$L =$ panel length in feet
$H =$ column height in feet
$D =$ column diameter in inches
$w =$ width or length of plate in inches
$t =$ thickness of plate in inches
$I_s =$ moment of inertia of column band
$I_c =$ moment of inertia of column section

$\frac{I_s}{L} =$ rigidity factor of slab
$\frac{I_c}{H} =$ rigidity factor of column
$R =$ Ratio of Rigidity Factors
$f'_c =$ ultimate compressive strength of the concrete in lbs. sq. inch
$n =$ total number of shear rods above plate on four sides
$a_s =$ area of each rod in sq. inches

Then in the case of the square panel

$$w = 8.35 \frac{W}{jd. f'c} \qquad t = \frac{W}{40{,}000\, D \times w} \qquad t^2 = \frac{R.\,M}{4{,}000\, w} \qquad a_s = \frac{W}{20{,}000\, n}$$

In the case of the rectangular panel, the head plate should also be rectangular in shape, the thickness being designed for the maximum column moment and band reaction and the length and width of the plate for the respective reactions in each direction.

INNER BAND DESIGN

The suspended inner bands are designed as a two-way panel supporting ¼ of the panel load distributed in two directions. In the square panel, ⅛ W is supported in each direction with the center of support assumed at the center of the half-column bands in each panel. The resulting inner band moment in each direction is therefore (plus or minus) 1/96 W.L. The inner bands could be entirely eliminated without destroying the structural integrity of the primary building frame. Openings in the floor plate therefore should be located, insofar as practicable, in the center half of the panel. If openings are located in the main column bands, special reinforcements or framing must be provided to restore the integrity of the band section. In any case, all large openings should have double reinforcement around the periphery with short diagonals at the corners of rectangular or square openings.

In the example illustrated, the inner bands are designed for a moment of 27 foot kips requiring six ¾″ or eight ⅝″ continuous rounds in each direction. The inner band rods need not be sharp-bended but might well be draped toward the center of the panel as a suspension system. In any event, they must be lapped sufficiently at all breaks in the reinforcement to develop the full area of the rod.

ADAPTABILITY

In the Williamsburgh Housing, the concrete slab with 3″ channel heads was 6 inches in thickness on spans varying from 11 to 19 feet, designed for a live load of 40 lbs. plus 15 lbs. per square foot for partitions. The Museum of Modern Art required 9 inch and 10 inch slabs with 6 inch channels fabricated from bent plates on 24 foot span for live loads of 100 and 150 lbs. per square foot. The 7 inch slabs with 3″ channel heads in the Leopold Residence have a maximum span of 20 feet.

When required by larger loads, greater height of building or length of span, LALLY COLUMNS can be reinforced as shown in Figs. 11 and 12, or the columns can be fabricated from extra heavyweight pipe or from higher yield strength steels. In some instances, the cluster-column of 2, 3 or more pipes welded into a unit can be adapted to the architectural design, to secure greater rigidity of section or to be concealed at the intersection of 2-, 3- or 4-way interior partitions.

ADVISORY SERVICES

In the interests of conservation of critical materials which are now in short supply and which are required for essential defense construction and accessory housing in the present national emergency,

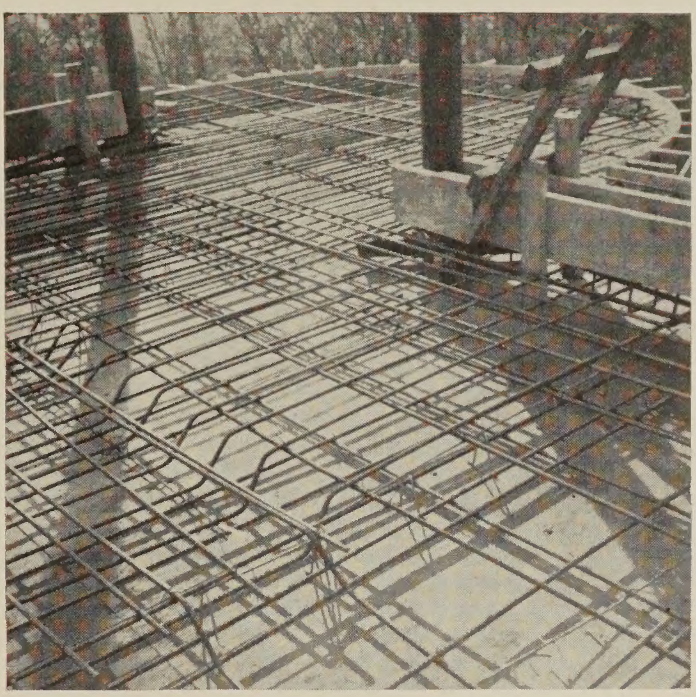


FIG. 27. CANTILEVER BALCONY—LEOPOLD RESIDENCE

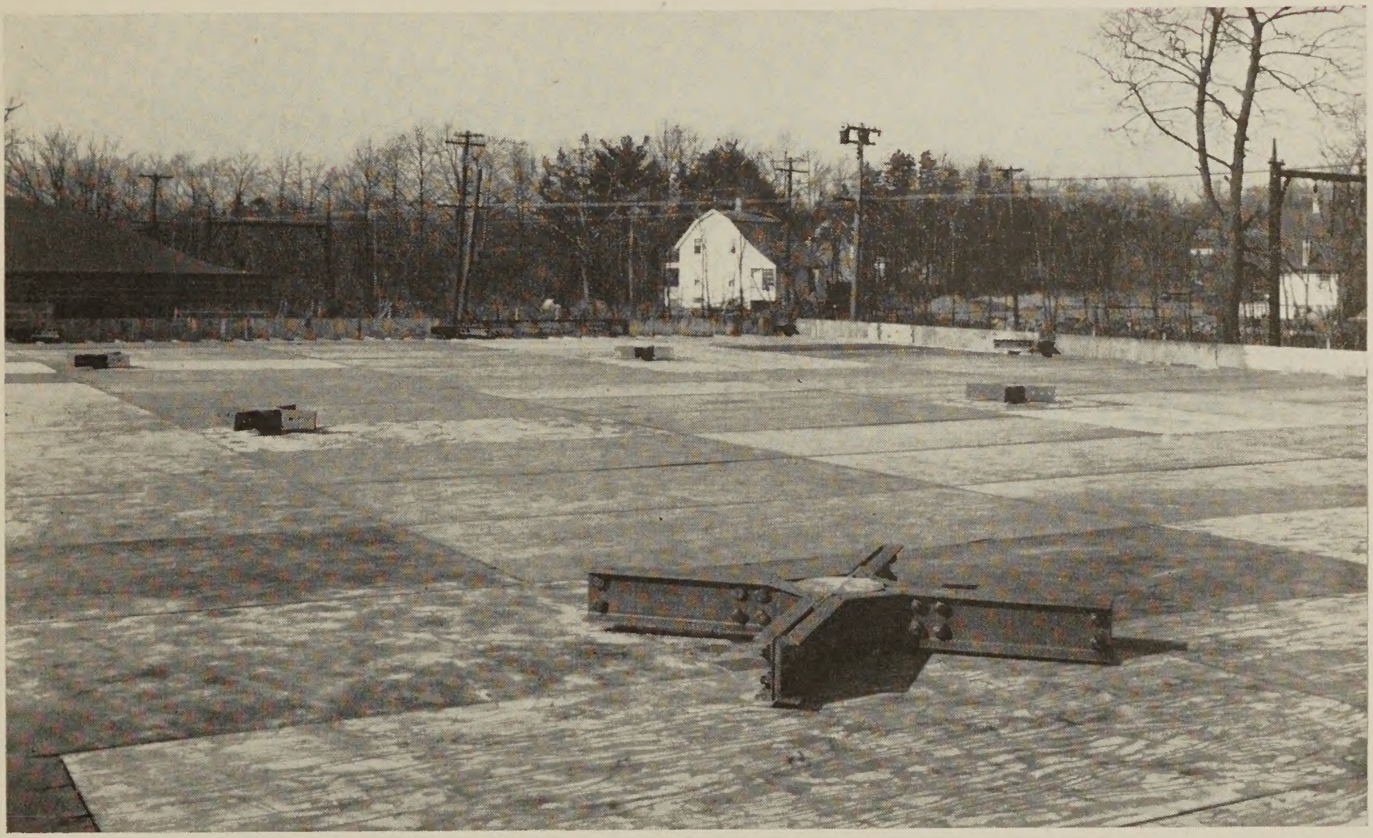
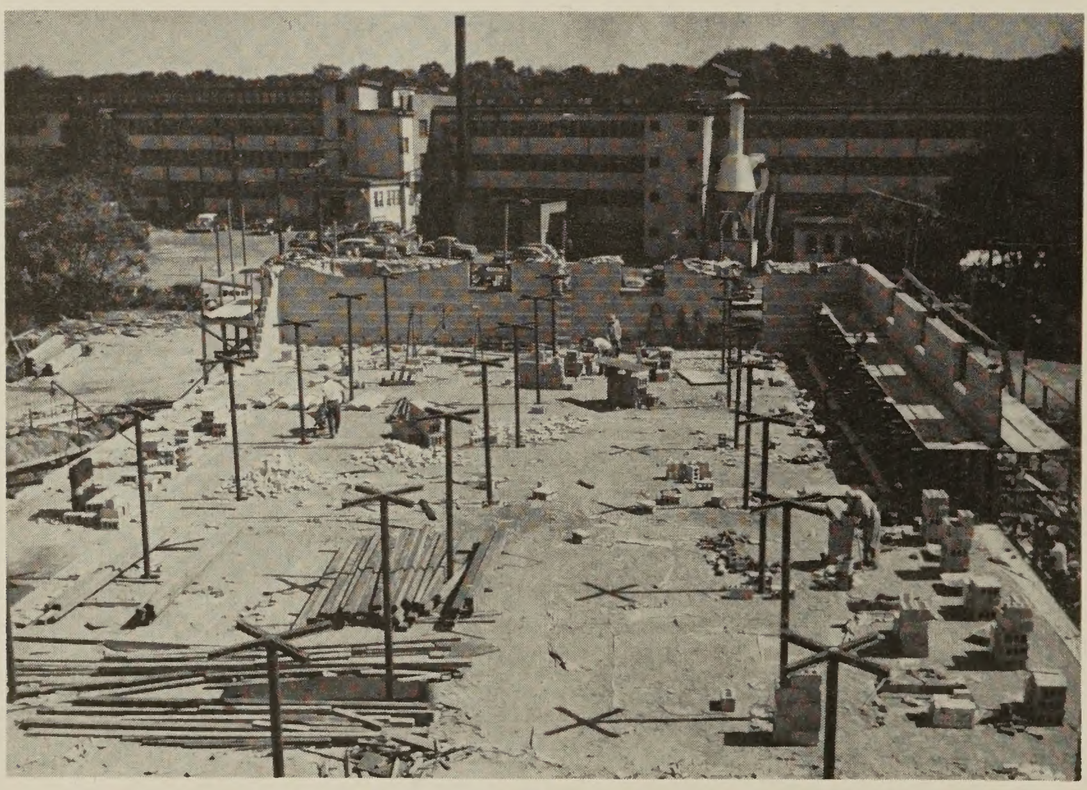


FIG. 28. HOOPER HOLMES OFFICE BUILDING
SIMPLICITY OF FORMWORK—COLUMN HEADS IN PLACE

the use of the LALLY COLUMN in conjunction with the EQUIDEPTH SYSTEM offers a timely and economical solution. The LALLY COLUMN COMPANY will furnish further technical data and cost estimates if required for specific applications of this form of construction in housing or commercial buildings on request.



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